



An analytical study of the behavior of composite girder bridges subjected to loads applied parallel to the plane of the slab
by Jagannath Kishanchand Khanna

A thesis submitted to the Graduate Faculty in partial fulfillment of the requirements for the degree of DOCTOR OF PHILOSOPHY in Civil Engineering and Engineering Mechanics
Montana State University
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Abstract:

The behavior of composite girder "bridges subjected to loads applied parallel to the plane of the slab was investigated. The finite element method of analysis which treats the slab and the longitudinal girders as an assemblage of plate elements was used. The force displacement relationship for the rectangular plate element was developed with six degrees of freedom, three translations and three rotations.

The series method of substructures is explained for solving a large number of simultaneous equations.

The influence of three kinds of diaphragms, beam, bar and plate diaphragms, on the behavior of the bridge was examined. From the analysis it was noted that the bridge undergoes considerable warping in the absence of diaphragms. The beam diaphragms, whose nodes coincide with the nodes of the slab, do not prevent the distortions of the bridge cross section. The bar and plate diaphragms are of great significance in reducing the transverse deflections of the bridge.

The transverse bending moments resulting from the vertical deflections of the girders are sizeable in the absence of bar or plate diaphragms.

The intermediate diaphragms are seen to have great importance in transferring the load from the loaded exterior girder to the unloaded girders when the loads are applied on the bottom edge of the exterior girder.

AN ANALYTICAL STUDY OF THE BEHAVIOR OF COMPOSITE GIRDER BRIDGES
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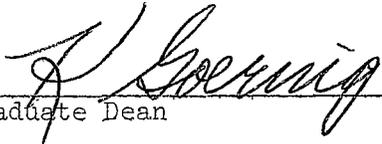
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THESES

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To

my wife Kaushal

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ABSTRACT

The behavior of composite girder bridges subjected to loads applied parallel to the plane of the slab was investigated. The finite element method of analysis which treats the slab and the longitudinal girders as an assemblage of plate elements was used. The force displacement relationship for the rectangular plate element was developed with six degrees of freedom, three translations and three rotations.

The series method of substructures is explained for solving a large number of simultaneous equations.

The influence of three kinds of diaphragms, beam, bar and plate diaphragms, on the behavior of the bridge was examined. From the analysis it was noted that the bridge undergoes considerable warping in the absence of diaphragms. The beam diaphragms, whose nodes coincide with the nodes of the slab, do not prevent the distortions of the bridge cross section. The bar and plate diaphragms are of great significance in reducing the transverse deflections of the bridge.

The transverse bending moments resulting from the vertical deflections of the girders are sizeable in the absence of bar or plate diaphragms.

The intermediate diaphragms are seen to have great importance in transferring the load from the loaded exterior girder to the unloaded girders when the loads are applied on the bottom edge of the exterior girder.

NOTATION

a, b	plate dimensions in the x and y directions, respectively
A	cross sectional area of bar diaphragms
[A]	coordinate transformation matrix relating element displacements to displacements of the overall structure
A_s	cross sectional area of eccentric beam diaphragms
c	a constant = $(1-\gamma)/2$
[C]	matrix relating dimensionless degrees of freedom with generalized coordinates
C_m	moment coefficient
[D]	matrix of Hooke's coefficients
E	modulus of elasticity
{f}	displacement field described in terms of generalized coordinates
{F}	force vector
G	modulus of rigidity
I	total potential energy
I_x	moment of inertia of beam cross section with respect to the reference surface
J	St. Venant's torsional constant
[k]	element stiffness matrix
[\underline{k}]	dimensionless stiffness matrix
[K]	stiffness matrix for the overall structure
L	span length of a bridge
M_x, M_y	moments about x and y axes, respectively

N_x, N_y	axial forces per unit length of plate
{P}	equivalent nodal forces
{P(x,y)}	matrix of displacement patterns in cartesian coordinates
{P(ξ, η)}	matrix of displacement patterns in dimensionless coordinates
{Q}	matrix of forces acting on an element
R_x, R_y, R_z	longitudinal, transverse and vertical reactions, respectively
S_x	first moment of the stiffener area with respect to the reference surface
[t]	transfer matrix relating dimensionless degrees of freedom $\{\delta_i\}$ at a node i of the plate element with dimensional degrees of freedom $\{\delta_i\}$
[T]	strain transfer matrix relating dimensional strains to dimensionless strains
u, v, w	displacement patterns in x, y and z directions, respectively
$\underline{u}, \underline{v}, \underline{w}$	displacement patterns in ξ, η and z directions, respectively
u_x, v_x, w_x	partial derivatives of u, v and w with respect to x
u_ξ, v_ξ, w_ξ	partial derivatives of u, v and w with respect to ξ
u_y, v_y, w_y	partial derivatives of u, v and w with respect to y
u_η, v_η, w_η	partial derivatives of u, v and w with respect to η
U, V, W	translational degrees of freedom
x, y, z	cartesian coordinate axes
{ α }, { β }	generalized coordinates
{ δ }	nodal degrees of freedom
{ $\underline{\delta}$ }	dimensionless degrees of freedom
ΔL	length of beam element

$\{\epsilon\}$	strain vector $(\epsilon_x, \epsilon_y, \epsilon_{xy})$
$\{\underline{\epsilon}\}$	dimensionless strain vector $(\epsilon_\xi, \epsilon_\eta, \epsilon_{\xi\eta})$
ξ, η	dimensionless coordinate axes
$\theta_x, \theta_y, \theta_z$	rotations about x, y and z axes, respectively
γ	Poisson's ratio
$[\Phi(\xi, \eta)]$	displacement patterns in terms of degrees of freedom
$\{\sigma\}$	column vector of stresses $(\sigma_x, \sigma_y, \tau_{xy})$

INTRODUCTION

1.1 GENERAL

Highway bridges consisting of stiffened steel plate or reinforced concrete slab decks supported by and acting compositely with girders have been used extensively during recent years. These bridges are subjected to loads applied perpendicular and parallel to the plane of the slab. The forces parallel to the plane of the slab include wind loads, earthquake loads and centrifugal forces. These lateral forces are normally ignored in the design of the deck slab and the main girders since their effects are thought to be minor compared to the effects of vertical loads. However, they are considered in the design of diaphragms and other lateral bracing between girders.

Existing composite girder bridges have proved satisfactory in their performance. However, it appears that a detailed analytical study of the spatial behavior of the composite girder bridges subjected to horizontal loads has never been undertaken in the past. The complete three-dimensional analysis of the bridge, therefore, can not be carried out using present day techniques.

A satisfactory analysis of a composite girder system can be accomplished by using the orthotropic plate theory provided the loads are applied vertically. When the girder system is subjected to loads applied parallel to the plane of the slab, the use of the orthotropic plate theory is highly questionable, since this type of loading produces torsion, and causes warping of the cross section.

1.2 OBJECT AND SCOPE

The purpose of this investigation is to develop an analytical model that can be used to study the response of composite girder systems subjected to loads applied parallel to the plane of the slab. In addition to studying the effects of these loads on the slab and main girders, it is planned to investigate the effect of various types of diaphragms on the structural behavior and load distribution characteristics of the bridge.

A finite element approach is used to study the behavior of composite girder bridges when subjected to lateral loads. The slab and web of the main girders are treated as an assemblage of plate elements. The force displacement relationship for the rectangular plate element having six degrees of freedom at each node (three translations and three rotations), is developed. Flanges of girders are treated as beams lying in the horizontal plane.

The influence coefficients for the deflections, shears and moments, etc., for composite girder bridges are obtained for the unit horizontal load. The loads are applied at the node points of the top and bottom flanges of the exterior girders. In order to analyze the role of diaphragms and to study the changes in the behavior of the bridges due to the inclusion of the diaphragms, the bridges are solved without diaphragms in one case and with diaphragms at the end sections of the bridges and at an interval of one-quarter span.

The study includes three kinds of diaphragms; beams, bars and plate

diaphragms. The beam diaphragms are treated as an assemblage of beam elements whose nodes coincide with the nodes of the middle plane of the slab. Plate and bar diaphragms are assumed to have nodes which coincide with the nodes of the top and the bottom flanges of the longitudinal girders.

The study also considers two different support conditions for the bridge. In one case the girders of the bridge are pinned at one end and are supported on rollers at the other end, whereas, in the second case both ends of the girders are pinned.

Various other parameters that affect the behavior of bridges, e.g., the relative stiffness of slab and longitudinal girders, number and spacing of longitudinal girders and diaphragms, are not included in the present investigation. Similarly the investigation does not consider the stability analysis of the bridge structures.

1.3 BACKGROUND

There is no literature available for analyzing the effects of horizontal loads on the composite girder bridges. A few techniques of analyses for the distribution of the vertical loads to the various longitudinal members forming the bridge are well known.

One method divides the structure into individual and longitudinal and transverse members, each possessing an appropriate flexural and torsional stiffness. For each point of intersection of the members, equations of deflections and slope compatibility are set up and a set of

governing simultaneous differential and/or algebraic equations is solved. Here, one could distinguish between the longitudinal or primary members of the structure from the secondary or transverse members by modifying the stiffness properties of various members. The works of Lightfoot and Sawko (1,2)*, Hendry and Jaegar (3), Hetenyi (4), Pippard and De Waele (5) are examples of the above method.

As is indicated by Davis, et al, (6), Newmark (7) developed a distribution procedure for application to slab on steel I-beams, wherein he assumed negligible shear transfer of longitudinal shear at the beam slab interface. He used the moment distribution method modified to include slab elements to distribute the transverse slab moments and shears. In this technique the flexural rigidity of the girders could be adjusted to compensate for composite action of the slab with the supporting girders.

A more rigorous method of analysis considers a composite beam bridge as an elastically equivalent slab system, an "orthotropic plate", whose structural properties in the two orthogonal directions are uniformly distributed along their length. Analyses of the orthotropic plates are generally based on Huber's (8) theory of anisotropic plates. Such a simplification of the bridge may be justified if (a) the ratios of stiffener spacing to slab boundary dimensions are very small to ensure approximate homogeneity of stiffness, (b) flexural and torsional rigidities are independent of the boundary conditions of the slab and the distribution of the load, (c) perfect bond exists between the slab and eccentric stiffeners, and

* Numbers in parentheses refer to references.

(d) there is no warping of the cross section of the bridge structure.

The idea of applying the theory of orthotropic plates to a grid system of a bridge deck by treating it as an idealized plate was proposed by Guyan (9) in 1946. Later, Massonnet (10, 11, 12), Cornelius (13), Pfluger (14, 15), Trenks (16), Glencke (17), and others extended and generalized the use of this method. Pfluger developed a system of three fourth order differential equations in order to include in-plane motions of the plate. Trenks showed that three simultaneous differential equations expressing three components of deformation of the deck plate may be transformed into one differential equation of eighth order. Most of the research published in recent years involves theoretical studies of orthotropic deck plates and deals with mathematical methods for analysing such structures (18 - 24).

A primary drawback of the orthotropic plate method is that the stiffnesses of the slab and beam are to be 'smeared' into an equivalent plate. The flexural and torsional rigidities of the plate can, at best, be approximations and are often difficult to evaluate. Secondly, once the solution of the plate problem is obtained, it is difficult to isolate the moments, shears and displacements for the longitudinal and transverse girders, which are of primary importance.

FINITE ELEMENT METHOD OF ANALYSIS

2.1 GENERAL

In the present investigation the finite element idealization is used as the basic numerical technique to solve a set of differential equations of a continuum with regard to appropriate boundary conditions. Engineering structures built up of bars, beams and plates are generally too complex to be analyzed by the theory of elasticity. The problem becomes tractable if the fundamental conditions of equilibrium and compatibility are expressed in such a manner that the mathematical formulation is given in terms of algebraic equations. The finite element technique is a convenient scheme for obtaining these equations.

The finite element method is very well described in the literature (25,26) and hence only a brief description of the general features of the method are given here. In addition, certain features of the present study which have not been presented before are discussed in detail.

The finite element method is divided into three steps:

Step 1: Structural Idealization:

A structural system is considered as an assemblage of discrete structural elements interconnected at a limited number of node points, usually the corners of the elements. Each element is assumed to have only a finite number of degrees of freedom. The formulation of such a model, referred to as the structural idealization, reduces the infinite degrees of freedom of the continuum to finite degrees of freedom suitable for the

matrix method of analysis. An engineering judgement is vital at this stage since an exact analysis is performed on this substitute structure and hence the results are valid only to the extent the substitute structure represents the original structure.

Step 2: Evaluation of Element Properties:

The force-displacement relationship - stiffness or flexibility matrix - which must be obtained now is the critical phase of the method. An element which has infinite degrees of freedom is restricted to limited degrees of freedom. This in general implies violation of the continuity conditions or the equilibrium conditions or both. Both compatibility and equilibrium conditions are satisfied only under certain special situations, like the case of bending of beams with two degrees of freedom (slope and deflection) at either end. Most of the elements reported in the literature are based on the displacement method (use of stiffness matrix) of analysis and may be subdivided into the following categories:

- (1) Elements satisfying displacement compatibility,
- (2) Elements satisfying equilibrium, or
- (3) Elements violating both equilibrium and compatibility

(1) Elements satisfying displacement compatibility:

A set of deformation patterns are chosen to define, uniquely the state of displacement within each element. The nodal displacements act as the undetermined parameters. Selection of the displacement patterns which will explicitly specify continuity of deformations and their first derivatives between adjacent elements is difficult. However, elements satisfying continuity of displacements and

their first derivatives and those which satisfy compatibility of displacements only, along the whole interface between adjacent elements are used in the literature with a great success.

Once the displacement patterns are chosen in terms of nodal degrees of freedom, the principle of virtual displacement or the principle of minimum total potential energy is employed to obtain force displacement relationship for the element. Hence the stiffness matrix for the element is obtained.

The process guarantees equilibrium of nodal forces, but does not ensure the stress equilibrium within the element or along the boundaries of the element unless the displacement functions are chosen so that they identically satisfy the differential equation(s) of equilibrium. The elements satisfying the displacement compatibility along the edges of the adjacent elements provide a lower bound to the correct solution. The value of such a bound is not very great since the underestimation of displacements and stresses is true only in an average sense over the entire continuum and is not necessarily true at every point of the continuum. It may be noted that the deformation patterns are invented rather than derived. Great care is required in choosing functions so that the necessary rigid body displacements are satisfied to ensure convergence to the correct solution.

(2) Elements satisfying equilibrium:

In this technique one assumes stress patterns which are

in equilibrium at the outset instead of assuming displacement distribution as in the previous method (27, 28). Now the virtual force approach or the principle of minimum total complementary potential energy is employed to obtain the stiffness matrix of the element. Alternately the displacement distribution, in terms of nodal degrees of freedom, may be derived from the assumed stress distribution and the method of virtual displacement may be employed for the calculation of the stiffness matrix.

The displacement distribution so obtained, in general, violates the compatibility of boundary displacements on adjacent elements. The method gives an over-estimate of the total strain energy and therefore provides an upper bound on the average displacements. Any distribution of strain can be approximated by uniform strains by reducing the element size. Hence, for convergence to the correct solutions it is essential to include stress patterns which produce uniform strain. The method is, in general, more difficult to derive than the previous method.

(3) Elements violating both equilibrium and compatibility:

Here, the displacement pattern is prescribed along the edges of the element in terms of its nodal values in such a way that complete compatibility between adjacent elements is established. The elasticity problem of the element subjected to these boundary displacements is solved, exactly or approximately. If an exact solution is obtained, an element which satisfies both compatibility and equilibrium conditions is obtained. For an approximate solution the complementary strain energy,

defined in terms of the internally equilibrating stress field, may be minimized. This violates the compatibility conditions within the element and the equilibrium conditions are satisfied only approximately on the boundaries. Experience with this technique has indicated good convergence (29).

Step 3: Analysis of the Element Assemblage:

Once the stiffness matrix for each element of the substitute structure is computed in its local coordinate system, it is modified into a global (entire structure) coordinate system. The elements of the modified stiffness matrix are placed in their correct positions in the larger framework of the stiffness of an entire structure. The overlapping terms are superimposed. This process is equivalent to carrying out the matrix multiplication $[A^T] [K][A]$. Where $[A]$ is a coordinate transformation matrix and $[K]$ is a square matrix with the stiffness matrix of each element listed on its main diagonal (26). In practice, the matrix multiplication is seldom carried out since it is time consuming and takes considerable computer core storage.

A necessary criterion of the assembly is that the degrees of freedom for the node of an element be equal to the degrees of freedom of the node of the structure. This may require expansion of the element stiffness matrix by inserting an appropriate number of zeroes. The general process of assembly for these stiffnesses are identical, irrespective of number of nodes an element possesses.

The stiffness matrix for the entire structure is a singular matrix

because the system is free to move as a rigid body when external loads are applied. The order of singularity of the matrix is equal to the number of possible rigid body motions. If the order of singularity is greater than this, then the structure is internally unstable or collapsible. A nonsingular matrix is now obtained by imposing sufficient boundary restraints on the structure.

It may be noted that forces acting on the structure are limited only to the nodes. If any other type of loads are applied to the structure then they must be reduced to "equivalent nodal forces" in the finite element analysis.

After the stiffness matrix for the structure is assembled, the simultaneous linear equations are ready for solution. Any standard method of solution of simultaneous equations may be employed. Certain special techniques may be used to solve a large number of simultaneous equations (see Chapter 3).

2.2 FINITE ELEMENT MODEL FOR BRIDGE STRUCTURE

The finite element approach is employed to treat a typical composite floor system (Figure 1) as a three dimensional space structure. The slab and girder elements are treated as an assemblage of rectangular plate elements each of its nodes having six degrees of freedom. Three degrees of freedom describe the transverse bending of the plate element and the remaining

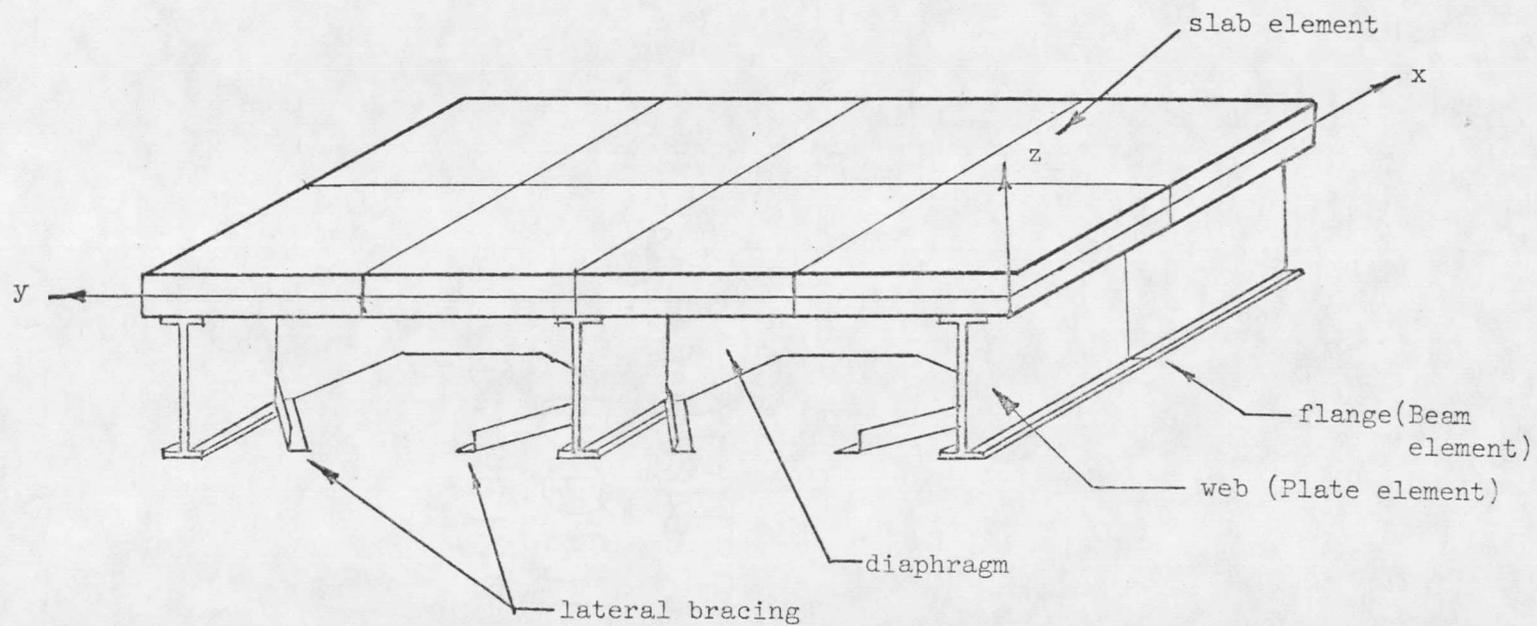


Figure 1: Typical Composite Girder System.

three degrees of freedom describe in-plane deformations of the plate. The flanges of the girders are treated as beams lying in a horizontal plane. Lateral bracing, if any, is treated as bars having only extensional stiffness.

2.2.1 Development of Stiffness Matrices for the Elements.

2.2.1.1 A Plate Element:

For a plate element the in-plane and bending deformations are treated as uncoupled for small deflections, and consequently, elastic properties can be evaluated separately for the in-plane and out-of-plane forces. The stiffness matrices for the two cases; in-plane stretching and transverse bending, are obtained independently, and the stiffness matrix for the total nodal degrees of freedom is obtained simply by combining these stiffnesses.

Consider a plate element ijkl, lying in the x-y plane, as shown in Figure 2. At each node six degrees of freedom are permitted. Three of them are translations U, V and W in the directions of the x, y, and z axes, and three are rotations θ_x , θ_y , and θ_z about the x, y, and z axes respectively. Positive directions of the rotations are determined by the right-hand screw rule and are shown by vectors directed along these axes.

For the element,

$$\theta_x = w_y$$

$$\theta_y = -w_x$$

$$\text{and, } \theta_z = (v_x - u_y) / 2$$

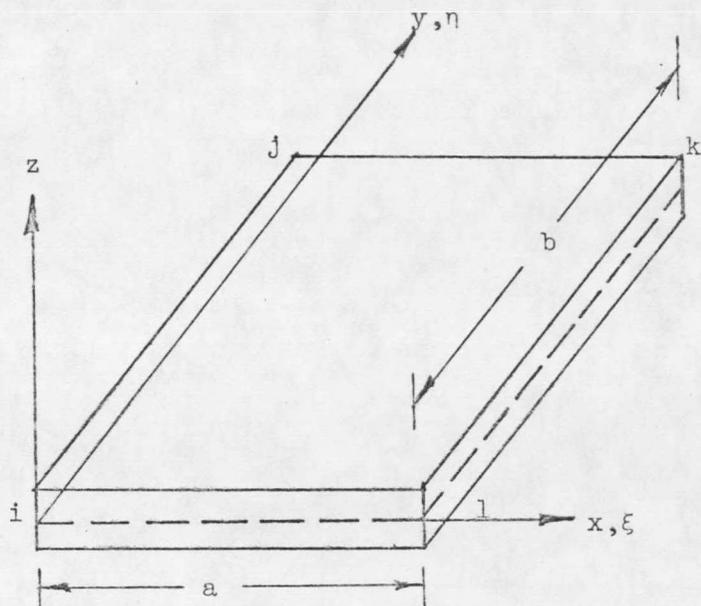
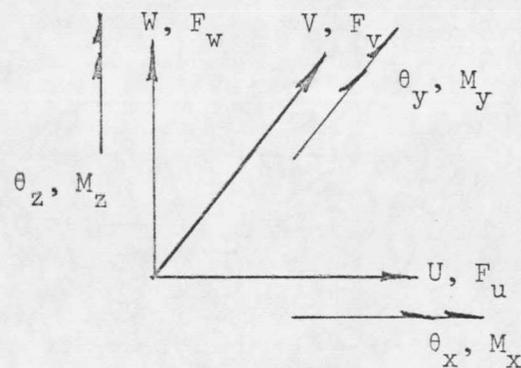


Plate Element



Sign Convention

Figure 2. Plate Element and Coordinate System

where w_y is the partial derivative of transverse deflection w with respect to y , w_x is the partial derivative of w with respect to x , and v_x and u_y are partial derivatives of v and u with respect to x and y , respectively.

Several rectangular plate elements having three degrees of freedom W , W_x and W_y are reported in the literature. Clough (30) has compared some of these elements and has presented a plate bending element which is a "best" element, both in terms of convergence and in terms of approximating true displacements and moments in the plate. The stiffness matrix of this element is used for the transverse bending stiffness of the plate element in the present analysis. The element is based on twelve generalized displacement patterns ($1, x, y, x^2, xy, y^2, x^3, x^2y, xy^2, y^3, x^3y, xy^3$). It is observed that the element satisfies equilibrium and displacement compatibility along the edges of the plate element whereas it does not satisfy the slope continuity along the boundaries.

On the other hand, there is no plane stress element described in the literature with U, V and θ_z degrees of freedom at its nodes.* In the past, elements with only two degrees of freedom, U and V , have been used. In the case of shell problems, where the third degree of freedom is desired, the corresponding stiffness matrix is obtained by inserting an appropriate number of zeroes into the stiffness matrix (25, p. 126; 31, p. 133). This not only is a misrepresentation of the element

* Triangular plate elements with six degrees of freedom $U, V, \epsilon_x, \epsilon_y, \gamma_{xy}$ and θ_z , and U, V, U_x, U_y, V_x and V_y at each node are presented in references 32 and 33.

stiffness but it also results in a singular matrix if all the elements should lie in one plane.

Development of a plate element with the above three degrees of freedom is of great engineering importance since it can be used for the case of concentrated moments and for the case of in-plane rotations, which are frequently encountered when two flat plates meet at an angle as in the case of folded plates, or when continuously curved surfaces of shells are approximated by flat plate segments. In folded plates and shells the element is subjected, generally, to both bending and in-plane forces. A detailed derivation of the stiffness matrix for the in-plane stretching of a rectangular plate element is presented.

The general procedure for obtaining the stiffness matrix for an element with assumed displacement patterns - with nodal degrees of freedom as unknown parameters - is described in section 2.1. The method is adopted to obtain the stiffness matrix of the plane stress element in the following two sections.

Section 2.2.1.2 is devoted to the selection of the displacement patterns. In this section the displacement patterns at first are assumed in terms of the generalized coefficients $\{\alpha\}$ and later are related to the nodal degrees of freedom $\{\delta\}$.

In section 2.2.1.3 the principle of the minimum total potential energy is used to compute the stiffness matrix for the plate element for the displacement patterns of section 2.2.1.2.

2.2.1.2 Selection of Displacement Patterns.

There are three generalized displacements at each node, two translations and one rotation, and a total of twelve for the element. These twelve nodal displacements establish the boundary conditions for the function which is selected to describe the deformations within the region of the element. This permits the use of a displacement field $f(x,y)$ with twelve generalized coordinates $\{\alpha\}$. Since the displacements along the x and y axes are, in general, independent of each other and symmetric in nature, it is desired to select six displacement patterns for $u(x,y)$ and six symmetric displacement patterns to describe the $v(x,y)$ deformation. Polynomial functions are easy to deal with and hence are frequently used in the finite element analysis. In order to choose simple deformation patterns, the functions are restricted to third order polynomials. If all the third order polynomial terms are included then there are twenty generalized coordinates $\{\alpha\}$, ten for the $u(x,y)$ displacements and ten for the $v(x,y)$ displacements. The $\theta_z(x,y)$ deformations are computed from the expressions of $u(x,y)$ and $v(x,y)$. (See Equations 2.1 through 2.6.)

$$\{f(x,y)\} = \begin{Bmatrix} u(x,y) \\ v(x,y) \\ \theta_z(x,y) \end{Bmatrix} = [P(x,y)] \{\alpha\} \quad (2.1)$$

where: $[P(x,y)] =$

$$\left[\begin{array}{cccccccccccccccc} b & 0 & \frac{bx}{a} & 0 & \frac{by}{b} & 0 & \frac{bxy}{ab} & 0 & \frac{by^2}{b^2} & 0 & \frac{bx^2}{a^2} & 0 & \frac{bx^2y}{a^2b} & 0 & \frac{bxy^2}{ab^2} & \frac{bx^3}{a^3} & 0 & \frac{by^3}{b^3} & 0 \\ 0 & a & 0 & \frac{ay}{b} & 0 & \frac{ax}{a} & 0 & \frac{axy}{ab} & 0 & \frac{ax^2}{a^2} & 0 & \frac{ay^2}{b^2} & 0 & \frac{axy^2}{ab^2} & 0 & 0 & \frac{ay^3}{b^3} & 0 & \frac{ax^3}{a^3} \\ 0 & 0 & 0 & 0 & \frac{-1}{2} & \frac{1}{2} & \frac{-x}{2a} & \frac{y}{2b} & \frac{-y}{b} & \frac{x}{a} & 0 & 0 & \frac{-x^2}{2a^2} & \frac{y^2}{2b^2} & \frac{-xy}{ab} & 0 & 0 & \frac{-3y^2}{2b^2} & \frac{3x^2}{2a^2} \end{array} \right]^*$$

(2.2)

and $\{\alpha\}$ is a vector of generalized coordinates.

* This form of the polynomials is selected in order to obtain a simpler polynomial in dimensionless coordinates.

If $\xi = x/a$ and $\eta = y/b$, then:

$$[P(x,y)] = \begin{bmatrix} b & 0 & 0 \\ 0 & a & 0 \\ 0 & 0 & 1 \end{bmatrix} [P(\xi,\eta)] \quad (2.3)$$

where ;

$$[P(\xi,\eta)] = \begin{Bmatrix} \underline{u}(\xi,\eta) \\ \underline{v}(\xi,\eta) \\ \underline{\theta}_z(\xi,\eta) \end{Bmatrix} = \begin{bmatrix} 1 & 0 & \xi & 0 & \eta & 0 & \xi\eta & 0 & \eta^2 & 0 \\ 0 & 1 & 0 & \eta & 0 & \xi & 0 & \eta\xi & 0 & \xi^2 \\ 0 & 0 & 0 & 0 & \frac{-1}{2} & \frac{1}{2} & \frac{-\xi}{2} & \frac{\eta}{2} & -\eta & \xi \\ \xi^2 & 0 & \xi^2\eta & 0 & \xi\eta^2 & 0 & \xi^3 & 0 & \eta^3 & 0 \\ 0 & \eta^2 & 0 & \eta^2\xi & 0 & \xi^2\eta & 0 & \eta^3 & 0 & \xi^3 \\ 0 & 0 & \frac{-\xi^2}{2} & \frac{\eta^2}{2} & -\xi\eta & \eta\xi & 0 & 0 & \frac{-3\eta^2}{2} & \frac{3\xi^2}{2} \end{bmatrix} \quad (2.4)$$

and:

a and b are lengths of the element along the x and y axes respectively.

This implies that

$$\begin{Bmatrix} u(x,y) \\ v(x,y) \\ \theta_z(x,y) \end{Bmatrix} = \begin{bmatrix} b & 0 & 0 \\ 0 & a & 0 \\ 0 & 0 & 1 \end{bmatrix} \begin{Bmatrix} \underline{u}(\xi,\eta) \\ \underline{v}(\xi,\eta) \\ \underline{\theta}_z(\xi,\eta) \end{Bmatrix} \quad (2.5)$$

and that:

$$\{\delta_i\} = \begin{bmatrix} b & 0 & 0 \\ 0 & a & 0 \\ 0 & 0 & 1 \end{bmatrix} \{\underline{\delta}_i\} = [t]\{\underline{\delta}_i\} \quad (2.6)$$

Where $\{\delta_i\}$ is a column vector of the degrees of freedom at node i , (U_i, V_i, θ_{zi}) ; $\{\underline{\delta}_i\}$ is a column vector of dimensionless degrees of freedom $(\underline{U}_i, \underline{V}_i, \underline{\theta}_{zi})$ of node i ; and the transfer matrix $[t]$ is

$$[t] = \begin{bmatrix} b & 0 & 0 \\ 0 & a & 0 \\ 0 & 0 & 1 \end{bmatrix} \quad (2.7)$$

In order to relate dimensionless degrees of freedom $\{\delta\}$ with generalized coordinates $\{\alpha\}$, the coordinates of nodes i, j, k and l are substituted into the matrix $[P(\xi, \eta)]$. It may be observed that columns 3, 11, and 17; 7 and 13; 8 and 14; and columns 4, 12 and 18 are identical in the resulting matrix.* Hence, the polynomial terms corresponding to columns 11, 12, 13, 14, 17 and 18, though they are independent displacement patterns, may be discarded from the definition of the function $P(\xi, \eta)$. Hence,

$$[P(\xi, \eta)] = \begin{bmatrix} 1 & 0 & \xi & 0 & \eta & 0 & \xi\eta & 0 & \eta^2 & 0 & \xi\eta^2 & 0 & \eta^3 & 0 \\ 0 & 1 & 0 & \eta & 0 & \xi & 0 & \eta\xi & 0 & \xi^2 & 0 & \xi^2\eta & 0 & \xi^3 \\ 0 & 0 & 0 & 0 & \frac{-1}{2} & \frac{1}{2} & \frac{-\xi}{2} & \frac{\eta}{2} & -\eta & \xi & -\xi\eta & \xi\eta & \frac{-3\eta^2}{2} & \frac{3\xi^2}{2} \end{bmatrix} \quad (2.8)$$

Equation 2.8 contains fourteen polynomial terms to define the displacement field $\{f(\xi, \eta)\}$ of the plate element. Seven of the terms define $\underline{u}(\xi, \eta)$ displacements and the remaining seven define $\underline{v}(\xi, \eta)$ displacements.

* see Appendix 1, page 130.

Each element has twelve dimensionless degrees of freedom $\{\delta\}$ and fourteen generalized coordinates $\{\alpha\}$. In order to uniquely relate the generalized coordinates $\{\alpha\}$ with the nodal degrees of freedom $\{\delta\}$, only twelve polynomial terms in the definition of $[P(\xi, \eta)]$ are needed. Ideally we are interested in a symmetric and independent set of displacement patterns to define $\underline{u}(\xi, \eta)$ and $\underline{v}(\xi, \eta)$ which gives a 12×12 , nonsingular matrix. Therefore, one displacement pattern from each definition of $\underline{u}(\xi, \eta)$ and a corresponding function from the definition of $\underline{v}(\xi, \eta)$ must be discarded. In order to retain simple displacement patterns it is preferred to discard a third order term from the definition of $\underline{u}(\xi, \eta)$ and $\underline{v}(\xi, \eta)$. Realistically, an independent set of symmetric displacement patterns is not available when all second order polynomial terms are retained in the definition of $[P(\xi, \eta)]$. This is observed by a careful examination of matrix $[C]$ which relates the dimensionless degrees of freedom $\{\delta\}$ with the generalized coordinates $\{\alpha\}$. The matrix $[C]$ is obtained by substituting the dimensionless coordinates for the nodes i, j, k and l of the element in the matrix $[P(\xi, \eta)]$. The 10×10 principle minor of the matrix $[C]$ is nonsingular which indicates linear independence of the first ten vectors of the matrix $[C]$. On the other hand, the 12×12 square matrix obtained by omitting columns eleven and twelve, or by deleting columns thirteen and fourteen, of the matrix $[C]$ is singular. This proves the assertion that an independent symmetric set of functions does not exist when all the second order polynomial terms are retained in the definition of $[P(\xi, \eta)]$.

It is also noted that any combination of two vectors, other than those mentioned above, are linearly independent of the first ten vectors. This shows that a set of linearly independent, non-symmetric deformations are available to describe the deformations $\underline{u}(\xi, \eta)$ and $\underline{v}(\xi, \eta)$. Non-symmetry of shape functions implies non-isotropy of the material and hence is of little interest.

Upon examination of other combinations of the fourteen vectors of matrix [C], it is found that among the chosen displacement patterns there is no independent symmetric set available to describe the deformations $\underline{u}(\xi, \eta)$ and $\underline{v}(\xi, \eta)$. Hence, we note that an arbitrary choice of independent displacement patterns is inadequate to guarantee existence of an inverse of the matrix [C] which relates the generalized coordinates $\{\alpha\}$ to the dimensionless degrees of freedom $\{\delta\}$.

In order to define a symmetric set of shape functions, an interdependent set of displacement functions is required. This is easily done by retaining the first ten vectors of the matrix [$\underline{P}(\xi, \eta)$], and by replacing the last four columns by two columns which are linear combinations of columns eleven and twelve and columns thirteen and fourteen respectively. Hence,

$$[\underline{P}(\xi, \eta)] = \begin{bmatrix} 1 & 0 & \xi & 0 & \eta & 0 & \xi\eta & 0 & \eta^2 & 0 & -\xi\eta^2 & -\eta^3 \\ 0 & 1 & 0 & \eta & 0 & \xi & 0 & \eta\xi & 0 & \xi^2 & \xi^2\eta & \xi^3 \\ 0 & 0 & 0 & 0 & -\frac{1}{2} & \frac{1}{2} & -\frac{\xi}{2} & \frac{\eta}{2} & -\eta & \xi & 2\xi\eta & \frac{3}{2}(\xi^2 + \eta^2) \end{bmatrix} \quad (2.9)$$

Matrix $[C]$, obtained by substituting the coordinates of nodes i, j, k and l into the matrix $[P(\xi, \eta)]$, is easily inverted.

A dimensionless displacement field of the element $\{f(\xi, \eta)\}$, is related to the dimensionless degrees of freedom $\{\delta\}$ by:

$$\{f(\xi, \eta)\} = \begin{bmatrix} b & 0 & 0 \\ 0 & a & 0 \\ 0 & 0 & 1 \end{bmatrix} [P(\xi, \eta)] [C]^{-1} \{\delta\} \quad (2.10)$$

Equations 2.6 and 2.7 relate the dimensionless degrees of freedom $\{\delta\}$ with dimensional degrees of freedom $\{\delta\}$ as:

$$\{\delta\} = \begin{bmatrix} [t]^{-1} & 0 & 0 & 0 \\ 0 & [t]^{-1} & 0 & 0 \\ 0 & 0 & [t]^{-1} & 0 \\ 0 & 0 & 0 & [t]^{-1} \end{bmatrix} \{\delta\} \quad (2.11)$$

Substituting Equation 2.11 into Equation 2.10, we obtain the displacement patterns in terms of the nodal degrees of freedom.

$$\{f(\xi, \eta)\} = [t][P(\xi, \eta)][C]^{-1} \begin{bmatrix} [t]^{-1} & 0 & 0 & 0 \\ 0 & [t]^{-1} & 0 & 0 \\ 0 & 0 & [t]^{-1} & 0 \\ 0 & 0 & 0 & [t]^{-1} \end{bmatrix} \{\delta\} \quad (2.12a)$$

$$= [\phi(\xi, \eta)] \{\delta\} \quad (2.12b)$$

where:

$$[\Phi(\xi,\eta)] = [t][\underline{P}(\xi,\eta)][C^{-1}] \begin{bmatrix} [t]^{-1} & 0 & 0 & 0 \\ 0 & [t]^{-1} & 0 & 0 \\ 0 & 0 & [t]^{-1} & 0 \\ 0 & 0 & 0 & [t]^{-1} \end{bmatrix} \quad (2.13)$$

2.2.1.3 Determination of Stiffness for the Plate Element

The total potential energy I of the elastic system in Equation 2.14 is made up of two parts, (a) internal strain energy, and (b) the energy of loads acting at the nodes, or:

$$I = 1/2 \int_V \{\sigma\}^T \{\epsilon\} dv - \{\delta\}^T \{F\} \quad (2.14)$$

where:

$\{\epsilon\}$ = column vector of strains $(\epsilon_x, \epsilon_y, \gamma_{xy})$,

$\{\sigma\}$ = column vector of stresses $(\sigma_x, \sigma_y, \tau_{xy})$, and

$\{F\}$ = column vector of nodal forces $(F_{xi}, F_{yi}, F_{\theta zi} \dots \dots)$

From the definition of strains:

$$\{\epsilon\} = \begin{Bmatrix} \epsilon_x \\ \epsilon_y \\ \gamma_{xy} \end{Bmatrix} = \begin{bmatrix} \frac{\partial}{\partial x} & 0 & 0 \\ 0 & \frac{\partial}{\partial y} & 0 \\ \frac{\partial}{\partial y} & \frac{\partial}{\partial x} & 0 \end{bmatrix} [P(x,y)] \quad (2.15a)$$

$$= \begin{bmatrix} \frac{1}{a} \frac{\partial}{\partial \xi} & 0 & 0 \\ 0 & \frac{1}{b} \frac{\partial}{\partial \eta} & 0 \\ \frac{1}{b} \frac{\partial}{\partial \eta} & \frac{1}{a} \frac{\partial}{\partial \xi} & 0 \end{bmatrix} \begin{bmatrix} b & 0 & 0 \\ 0 & a & 0 \\ 0 & 0 & 1 \end{bmatrix} [\underline{P}(\xi,\eta)] \quad (2.15b)$$

$$\begin{aligned}
&= \begin{bmatrix} \frac{b}{a} & 0 & 0 \\ 0 & \frac{a}{b} & 0 \\ 0 & 0 & 1 \end{bmatrix} \begin{bmatrix} \frac{\partial}{\partial \xi} & 0 & 0 \\ 0 & \frac{\partial}{\partial \eta} & 0 \\ \frac{\partial}{\partial \eta} & \frac{\partial}{\partial \xi} & 0 \end{bmatrix} \begin{Bmatrix} \underline{u}(\xi, \eta) \\ \underline{v}(\xi, \eta) \\ \underline{\theta}_z(\xi, \eta) \end{Bmatrix} \quad (2.15c) \\
&= [\underline{T}]\{\underline{\epsilon}\}
\end{aligned}$$

where the strain transfer matrix

$$[\underline{T}] = \begin{bmatrix} \frac{b}{a} & 0 & 0 \\ 0 & \frac{a}{b} & 0 \\ 0 & 0 & 1 \end{bmatrix} \quad (2.15d)$$

and the dimensionless strain vector

$$\{\underline{\epsilon}\} = \begin{Bmatrix} \frac{u}{\xi} \\ v \\ \frac{u}{\eta} + \frac{v}{\xi} \end{Bmatrix} = [\underline{B}]\{\delta\} \quad (2.16)$$

The matrix $[\underline{B}]$ in Equation 2.16 is obtained by proper differentiation of the matrix $[\underline{\Phi}(\xi, \eta)]$ of Equation 2.13.

From the generalized Hooke's law, the stresses are:

$$\{\sigma\} = [D] \{\epsilon\} = [D] [T] \{\underline{\epsilon}\} = [D] [T] [\underline{B}] \{\delta\} \quad (2.17)$$

where:

$$[D] = \frac{E}{1 - \gamma^2} \begin{bmatrix} 1 & \gamma & 0 \\ \gamma & 1 & 0 \\ 0 & 0 & \frac{1-\gamma}{2} \end{bmatrix} \quad (2.18)$$

E in Equation 2.18 is the modulus of elasticity, and γ is Poisson's ratio for the material.

Substituting Equations 2.15, 2.16 and 2.17 into Equation 2.14 yields:

$$I = \frac{1}{2} \int_v ([D] [T] [\underline{B}] \{\delta\})^T ([T] [\underline{B}] \{\delta\}) dv - \{\delta\}^T \{F\} \quad (2.19a)$$

$$I = \frac{1}{2} \{\delta\}^T [k] \{\delta\} - \{\delta\}^T \{F\} \quad (2.19b)$$

$$\text{where } [k] = \int_v [\underline{B}]^T [T]^T [D] [T] [\underline{B}] dv \quad (2.20)$$

According to the principle of minimum potential energy

$$\frac{\partial I}{\partial \{\delta\}} = 0$$

$$\text{Hence: } [k] \{\delta\} - [F] = \{0\}$$

$$\text{or } [F] = [k] \{\delta\} \quad (2.21)$$

By definition, $[k]$ is the desired stiffness matrix.

Explicit expressions for the stiffness matrix are derived and presented in Appendix 1 along with the stiffness matrix for the

transverse bending of the plate element.

2.2.1.4 Limitations of the Plane Stress Finite Element

The deflection patterns chosen to represent the stretching of the plate do not completely satisfy the displacement continuity between adjacent elements. Continuity is maintained for the u displacement on edges parallel to the x axis and the v displacement on edges parallel to the y axis. The element exactly satisfies the constant strain situation. To verify this property of the element, uniform compression and uniform shear stresses were applied on a single element. The element is pin supported at one corner and is supported on rollers at the other end. The uniform stresses were reduced to equivalent nodal forces and were applied to the nodes of a single element. Plate elements with length to width ratios varying from 1 to 20 were examined. In all cases, nodal deflections were identical to corresponding deflections obtained with the theory of elasticity solutions. When stresses are not uniform, as in the case of a uniform bending stress, the resulting deflections are 6.25% too small. This disagreement indicates that in the case of linear stress distribution, or where stresses vary as a non-linear function, the plate must be taken as an assembly of small elements so that the distributed stress may be approximated by constant stresses.

Since the element correctly represents a uniform stress situation and it incorporates all the rigid body displacements, it is expected that the solution of plane stress problems obtained with the element will converge to the true solution when the element size is diminished. The

exact solutions to plane stress problems with rectangular boundaries are not easily available, and hence comparisons are limited to a cantilever beam loaded at its free end. It may be noted that the cantilever beam problem is not expected to reveal advantages of additional degrees of freedom of the new element, but is designed to show the convergence criterion.

Deflections of the cantilever beam for various mesh sizes are plotted in Figure 3. The figure also shows deflections of the free end when stiffness matrices proposed by Turner, et al (34) and Melosh (35) are used. Melosh's plate element assumes linear displacement patterns, whereas Turner's element is obtained from self equilibrating stress patterns. This element was found to give the best approximation to the cantilever beam problem by Hooley and Hibbert (36).

2.2.2 Beam Elements

As was mentioned in section 2.2 longitudinal girders are treated as plates in a vertical plane and their flanges are treated as beams lying in horizontal planes. If transverse beam diaphragms are provided in the bridge, they are treated as eccentric stiffeners which are rigidly connected to the slab. The stiffness matrix for such an eccentric stiffener, with its nodes in the middle plane of the slab, is based on the displacement equations:

$$u(x,y,z) = u(x,y,0) - zw_x$$

$$v(x,y,z) = v(x,y,0) - zw_y$$

where $u(x,y,0)$ and $v(x,y,0)$ are displacements of the reference surface

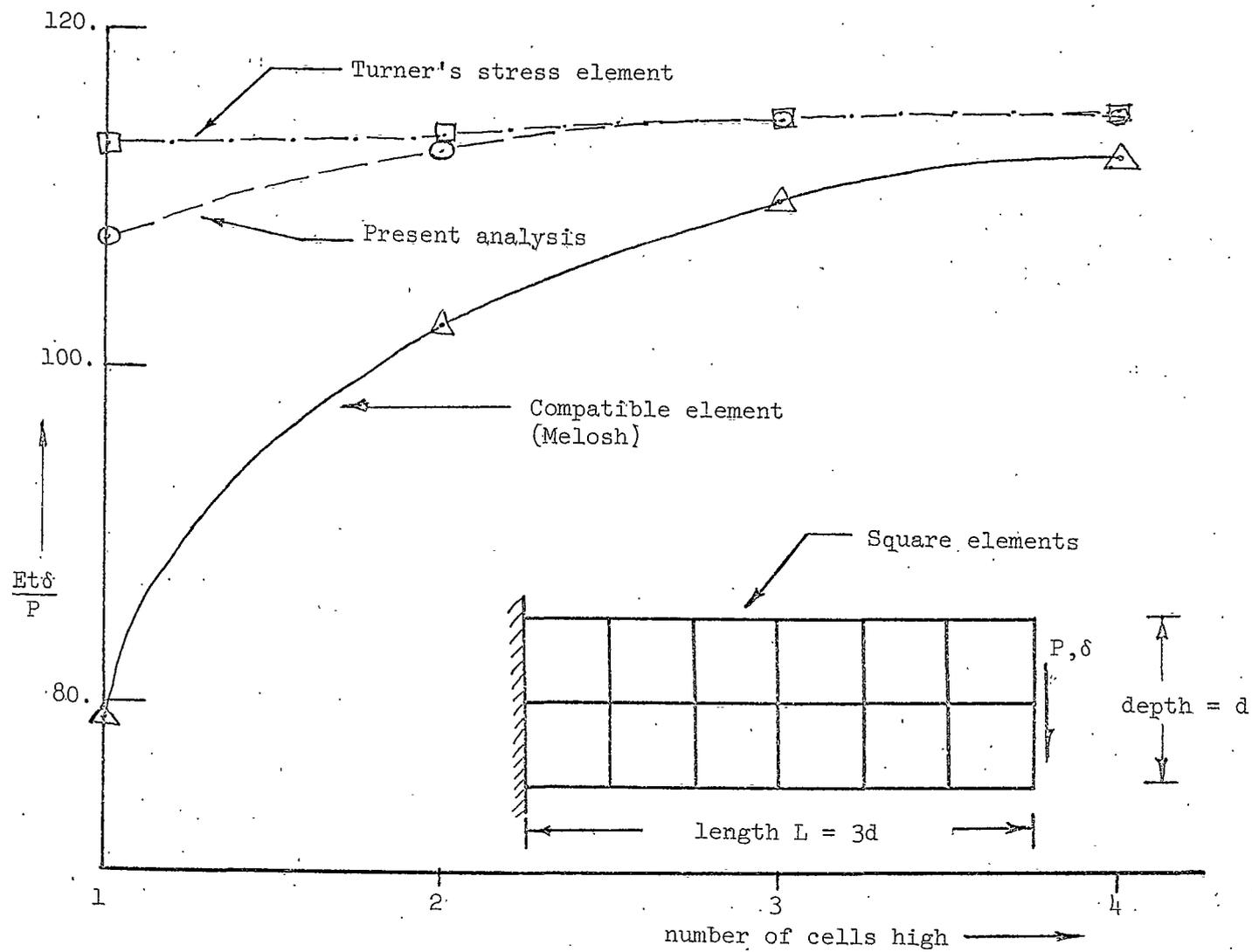


Figure 3. Convergence of finite element method of analysis.

along the x and y axes. These displacements are referred to as the u and v displacements.

If the beam lies entirely in the y-z plane, as is the case with transverse diaphragms, its resistance to motion in the x direction is completely ignored. The displacements in the y and z directions are assumed to be:

$$\begin{Bmatrix} v \\ w \end{Bmatrix} = \begin{bmatrix} 1 & y & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & y & y^2 & y^3 \end{bmatrix} \{\beta\} \quad (2.23)$$

Proceeding as in the case of the rectangular plate in extension, the generalized coefficients $\{\beta\}$ are related to degrees of freedom V, W, and $W_y (= \theta_x)$ of the beam element by:

$$\{\beta\} = [C^{-1}] \{\delta\} \quad (2.24)$$

where $[C]$ is 6x6 matrix of nodal coordinates.

The displacements in Equation 2.23 are represented in terms of the degrees of freedom $\{\delta\}$ of the beam element by:

$$\begin{Bmatrix} v \\ w \end{Bmatrix} = \begin{bmatrix} 1 & y & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & y & y^2 & y^3 \end{bmatrix} [C^{-1}] \{\delta\} \quad (2.25)$$

Therefore, the strain and stress vectors are given by:

$$\{\epsilon_y\} = v_y - zw_{yy} = (0 \ 1 \ 0 \ 0 \ -2z \ -6zy) [C^{-1}] \{\delta\} \quad (2.26)$$

and

$$\{\sigma\} = E\{\epsilon_y\} \quad (2.27)$$

The principle of minimum total potential energy gives the desired matrix equation:

In the above equations, A_s is the cross sectional area of the stiffener, S_x is the first moment of the stiffener area with respect to the reference surface, and I_x is the moment of inertia of the stiffener with respect to the reference surface.

In order to consider torsional resistance of the stiffener, extra degrees of freedom θ_y are introduced at both ends of the beam element. Assuming the angle of twist varies linearly along the length of the element, the twisting moments, F_{θ_y} , are related to the degree of freedom θ_y by:

$$\begin{Bmatrix} F_{\theta_{y1}} \\ F_{\theta_{y2}} \end{Bmatrix} = E \begin{bmatrix} \gamma & -\gamma \\ -\gamma & \gamma \end{bmatrix} \begin{Bmatrix} \theta_{y1} \\ \theta_{y2} \end{Bmatrix} \quad (2.29)$$

where $\gamma = GJ/E\Delta L$; G is the shear modulus, $J = \sum \frac{1}{3} bt^3$, and ΔL is the length of the beam element.

The torsional stiffness may be combined with the stiffness matrix of Equation 2.24 to obtain a stiffness matrix for the beam element having four degrees of freedom V , W , θ_x and θ_y at its nodes. The matrix is shown in Appendix 1. If the reference surface is assumed to pass through the neutral axis of the beam element then the stiffness matrix of the beam element may be reduced to a more familiar form by substituting $S_x = 0$ in the stiffness matrix.

2.2.3 Bar Elements

Bars or axial force members are used in the bridge as lateral

bracing or as diaphragms. The stiffness matrix of the bar element is obtained in local coordinates simply by assuming linear displacement patterns. If the direction of the axis of the bar is denoted by \bar{x} and the displacements and forces in this direction by \bar{u} and \bar{F}_u respectively, then the stiffness matrix for a bar element of length ΔL is given by:

$$\begin{Bmatrix} \bar{F}_{u1} \\ \bar{F}_{u2} \end{Bmatrix} = \frac{A E}{\Delta L} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{Bmatrix} \bar{U}_1 \\ \bar{U}_2 \end{Bmatrix} \quad (2.30)$$

where A is the area of cross section of the bar element.

2.3 EQUIVALENT NODAL FORCES

In the finite element analysis of the structure, loads are restricted to the nodes, whereas the orientation of the actual loads on the structure is arbitrary, e.g., truck loads on a bridge. Or, the loads are distributed over the structure as is the case with the dead load of the structure. In the finite element analysis one needs to replace the actual loads acting on the structure by the equivalent nodal forces. This replacement is easily done by equating the virtual work of the two systems:

$$\delta W = \int_s \delta\{f\}^T [Q] ds = \delta\{\delta\}^T \{P\}_{\text{equivalent}} \quad (2.31)$$

where $\delta\{f\}$ denotes the virtual displacement of the surface s and $[Q]$ is the matrix of the forces acting on the element, and $\delta\{\delta\}$ is the virtual displacements for the nodal degrees of freedom.

In Equation 2.12, $\{f\} = [\Phi]\{\delta\}$, hence:

$$\delta\{f\} = [\Phi]\delta\{\delta\} \quad (2.32)$$

Substituting Equation 2.32 in Equation 2.31,

$$\delta\{\delta\}^T \int_S [\Phi]^T [Q] ds = \delta\{\delta\}^T \{P\}_{\text{equivalent}} \quad (2.33)$$

Since virtual displacements $\delta\{\delta\}$ are arbitrary

$$\{P\}_{\text{equivalent}} = \int_S [\Phi]^T [Q] ds \quad (2.34)$$

To illustrate the method, equivalent nodal forces for the cases of uniform compression and uniform shear stress acting on the plane stress element of the previous section are computed. When the compressive stresses are acting on the edges parallel to y axis, the integration in Equation 2.34 is carried out only along these edges. The vector $[Q]$ of the applied loads is represented by two components, $\text{Col}^n(1,0,0)$ along $x = 0$ and $\text{Col}^n(-1,0,0)$ at $x = a$. On integration Equation 2.34 yields the equivalent nodal forces $\{P\}$ of Equation 2.35.

$$\begin{array}{l}
 \{P\} \\
 \text{compression}
 \end{array}
 =
 \begin{array}{c}
 \frac{b}{2} \\
 0 \\
 -\frac{b^2}{12} \\
 \frac{b}{2} \\
 0 \\
 \frac{b^2}{12} \\
 -\frac{b}{2} \\
 0 \\
 -\frac{b^2}{12} \\
 -\frac{b}{2} \\
 0 \\
 \frac{b^2}{12}
 \end{array}
 \quad (2.35)$$

Similarly, the equivalent nodal forces for the cases of uniform shear stresses and uniform bending stresses acting on edges parallel to y axis are obtained as shown in Equations 2.36 and 2.37, respectively.

{P}
shear =

$$\left[\begin{array}{c} \frac{-\sigma}{2} \\ \frac{-\sigma}{2} \\ 0 \\ \hline \frac{\sigma}{2} \\ \frac{-\sigma}{2} \\ 0 \\ \hline \frac{\sigma}{2} \\ \frac{\sigma}{2} \\ 0 \\ \hline \frac{-\sigma}{2} \\ \frac{\sigma}{2} \\ 0 \end{array} \right]$$

(2.36)

and

$$\{P\}_{\text{bending}} =$$

$$\begin{Bmatrix} p/b & 0 & 0 \\ 0 & 0 & 0 \\ p/b & 0 & 0 \\ 0 & 0 & 0 \\ p/b & 0 & 0 \\ 0 & 0 & 0 \end{Bmatrix}$$

(2.37)

COMPARISONS WITH EXISTING SOLUTIONS

3.1 INTRODUCTION

The displacement method of analysis when applied to the analysis of a structure composed of many elements, results in a large number of simultaneous algebraic equations. The conventional techniques adopted for the solution of these equations may cause computational difficulties or may even prove impossible when the order of the matrix exceeds the capacity of the digital computer.

In order to efficiently solve these equations on the computer, advantage must be taken of a high proportion of zero elements in the stiffness matrix. With careful numbering of the nodes, the stiffness matrix of the structure may be obtained so that all the non zero terms are located in a reasonably narrow diagonal band. The band width of the matrix is minimized when the nodes are numbered such that the difference between any two connected node numbers is a minimum. Several papers which deal with the banded form of the matrix have reported considerable economies, both in computing time and computer core storage (37, 38, 39, 40, 41).

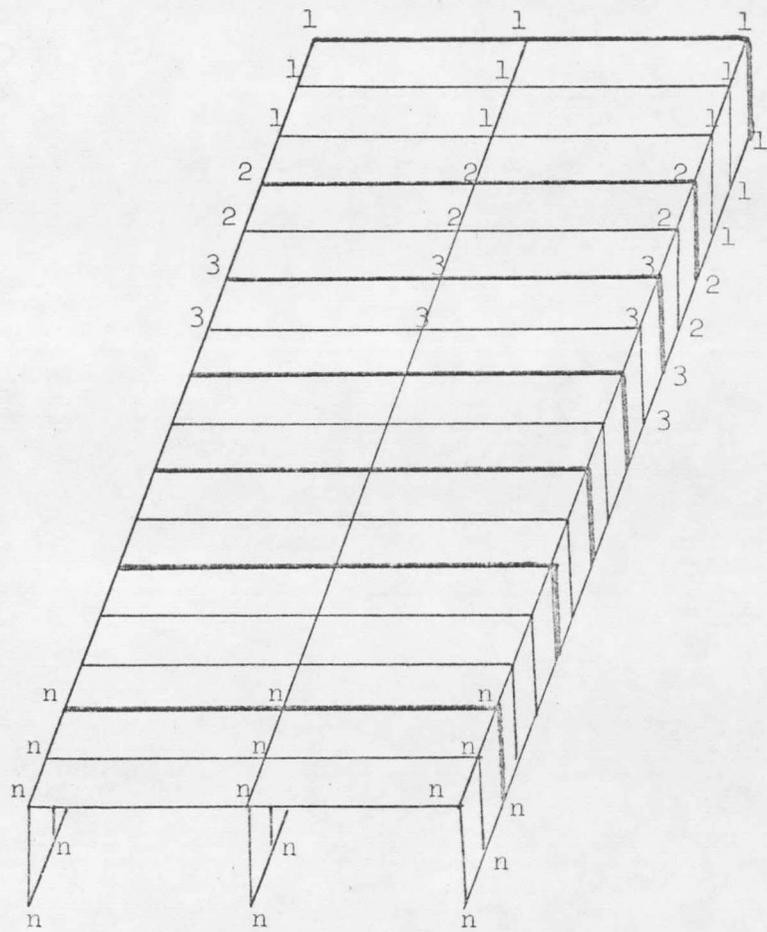
Alternately, the structure under consideration may be partitioned into a number of substructures, and the boundaries of the substructures may be specified arbitrarily. If the stiffness matrices for the substructures are determined, then each substructure can be treated as a complex structural element and the displacement method of analysis can be formulated for the partitioned structure. In other words, the idea of a

substructure is nothing more than a generalization of a structural member. Here the substructure is treated as a basic "building block" (42, 43, 44, 45). Such a physical idealization has a mathematical counterpart in the form of partitioned matrices.

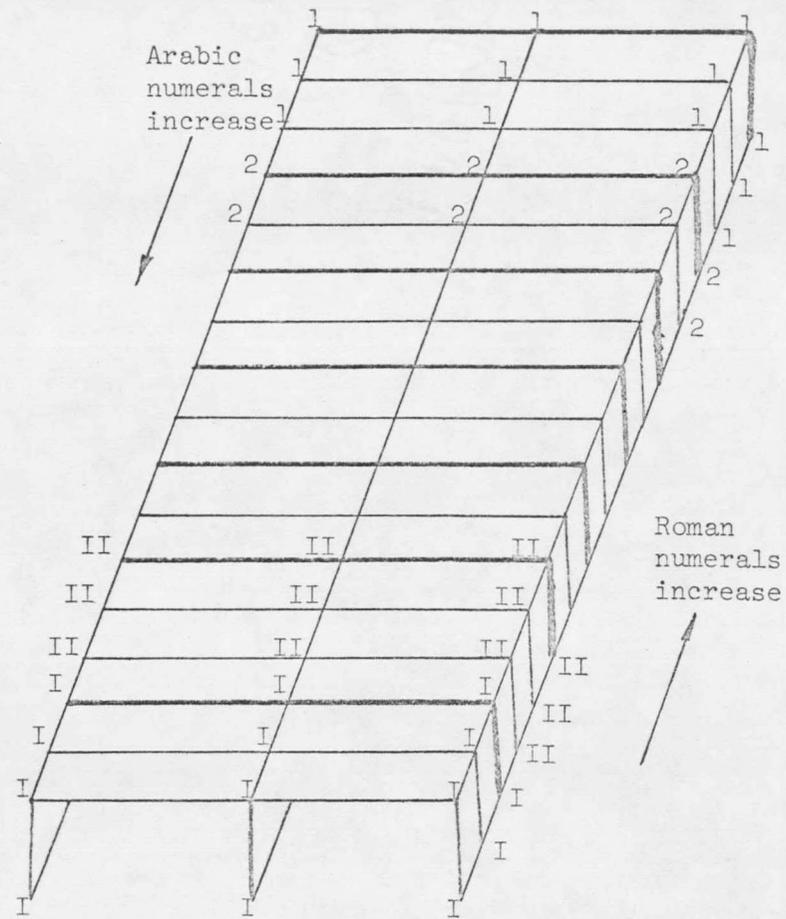
The complexity of the interactions between substructures depends primarily upon the manner in which the structure is divided. Moreover, the choice of the scheme for the solution of the resulting matrix equation is related to the layout of substructures. The method of series elimination is the simplest method among the various available techniques of subdivision of the structure. The method is explained briefly in the following section.

3.2 SERIES ELIMINATION METHOD OF SUBSTRUCTURE

When the structure is subdivided into 'n' substructures as shown in Figure 4(a), the stiffness matrix $[K]$ for the structure may be partitioned into a tri-diagonal band matrix, as given by:



(a)



(b)

Figure 4. Division of structure into n substructures.

$$\begin{bmatrix} [F]_1 \\ [F]_2 \\ \vdots \\ [F]_n \end{bmatrix} = \begin{bmatrix} [k]_{11} & [k]_{12} & & & \\ [k]_{21} & [k]_{22} & [k]_{23} & & \\ & [k]_{32} & [k]_{33} & [k]_{34} & \\ & & & & \\ & & & & [k]_{n-1,n} & [k]_{n,n} \end{bmatrix} \begin{bmatrix} [\delta]_1 \\ [\delta]_2 \\ \vdots \\ [\delta]_n \end{bmatrix} \quad (3.2)$$

In Equation 3.2, $\{F\}_j$ and $\{\delta\}_j$ refer, respectively, to the forces and displacements at the nodes of substructure j .

Equation 3.2 may be expanded to 'n' matrix equations. The first and second of these equations are:

$$[k]_{11} [\delta]_1 + [k]_{12} [\delta]_2 = [F]_1, \text{ and} \quad (3.3)$$

$$[k]_{21} [\delta]_1 + [k]_{22} [\delta]_2 + [k]_{23} [\delta]_3 = [F]_2 \quad (3.4)$$

From Equation 3.3,

$$[\delta]_1 = - [k]_{11}^{-1} \left[[k]_{12} [\delta]_2 - [F]_1 \right] \quad (3.5)$$

Substituting this expression for $[\delta]_1$ into equation 3.4:

$$[k]_{22}^* [\delta]_2 + [k]_{23} [\delta]_3 = [F]_2^* \quad (3.6)$$

where:

$$[k]_{22}^* = [k]_{22} - [k]_{21} [k]_{11}^{-1} [k]_{12}, \text{ and} \quad (3.7)$$

$$[F]_2^* = [F]_2 - [k]_{21} [k]_{11}^{-1} [F]_1 \quad (3.8)$$

From Equation 3.6:

$$[\delta]_2 = - [k]_{22}^{*-1} \left[[k]_{23} [\delta]_3 - [F]_2^* \right] \quad (3-9)$$

The form of Equation 3.9 is identical to the form of Equation 3.5.

The third of the matrix Equations, 3.2, may be written:

$$[F]_3 = [k]_{32} [\delta]_2 + [k]_{33} [\delta]_3 + [k]_{34} [\delta]_4 \quad (3-10)$$

Equations 3.6 and 3.10 form a pair of equations which are identical in form to the first pair of Equations 3.3 and 3.4. Operating on this pair, just as on the first pair, yields:

$$[k]_{33}^* [\delta]_3 + [k]_{34} [\delta]_4 = [F]_3^* \quad (3-11)$$

where:

$$[k]_{33}^* = [k]_{33} - [k]_{32} [k]_{22}^{*-1} [k]_{23}, \text{ and} \quad (3-12)$$

$$[F]_3^* = [F]_3 - [k]_{32} [k]_{22}^{*-1} [F]_2 \quad (3-13)$$

Equation 3.11 yields:

$$[\delta]_3 = - [k]_{33}^{*-1} \left[[k]_{34} [\delta]_4 - [F]_3^* \right] \quad (3-14)$$

One may now write the fourth of the matrix Equations 3.2. This equation and Equation 3.11 will form a third pair of equations. This pair of equations is similar in form to the first and second pair.

Proceeding as before one can write:

$$[k]_{44}^* [\delta]_4 + [k]_{45} [\delta]_5 = [F]_4^* \quad (3-15)$$

where :

$$[k]_{44}^* = [k]_{44} - [k]_{43} [k]_{33}^{*-1} [k]_{34}, \text{ and} \quad (3.16)$$

$$[F]_4^* = [F]_4 - [k]_{43} [k]_{33}^{*-1} [F]_3 \quad (3.17)$$

Proceeding in this manner, the general recursion relation is:

$$[k]_{j,j}^* [\delta]_j + [k]_{j,j+1} [\delta]_{j+1} = [F]_j^* \quad (3.18)$$

where:

$$[k]_{j,j}^* = [k]_{j,j} - [k]_{j,j-1} [k]_{j-1,j-1}^{*-1} [k]_{j-1,j}, \text{ and} \quad (3.19)$$

$$[F]_j^* = [F]_j - [k]_{j,j-1} [k]_{j-1,j-1}^{*-1} [F]_{j-1} \quad (3.20)$$

From Equation 3.18:

$$[\delta]_j = -[k]_{j,j}^{*-1} \left[[k]_{j,j+1} [\delta]_{j+1} - [F]_j^* \right] \quad (3.21)$$

The process is continued until the last pair of Equations, 3.22

and 3.23, are obtained:

$$[k]_{n-1,n-1}^* [\delta]_{n-1} + [k]_{n-1,n} [\delta]_n = [F]_{n-1}^* \quad (3.22)$$

$$[k]_{n-1,n} [\delta]_{n-1} + [k]_{n,n} [\delta]_n = [F]_n \quad (3.23)$$

From this pair of equations,

$$[\delta]_n = [k]_{n,n}^{*-1} [F]_n \quad (3.24)$$

where:

$$[k]_{n,n}^* = [k]_{n,n} - [k]_{n,n-1} [k]_{n-1,n-1}^{*-1} [k]_{n-1,n}, \text{ and} \quad (3.25)$$

$$[F]_n^* = [F]_n - [k]_{n,n-1} [k]_{n-1,n-1}^* [F]_{n-1} \quad (3.26)$$

Once the deflections $[\delta]_n$ of the last substructure are obtained, the deflections $[\delta]_{n-1}$ for the preceding structure are obtained from Equation 3.21 by substituting $j = n-1$. This process is repeated until the deflections $[\delta]_1$ for the first substructure are obtained.

The process just described computes the deflections of one end and then computes the deflections of the entire structure by using the recursion relation of Equation 3.21. Because of the large number of computations involved in the process it is suspected that a considerable loss in accuracy may result in the deflections of the structure. The error in computations may be reduced by first calculating the deflections of the midsection, and then, relating the deflections of the remaining structure with these deflections. In order to derive the recursion relations for this case, the substructures are numbered as shown in Figure 4(b). The resulting matrix equations are expressed by:

$$\begin{bmatrix} [F]_1 \\ [F]_2 \\ [F]_C \\ [F]_{II} \\ [F]_I \end{bmatrix} = \begin{bmatrix} [k]_{1,1} & [k]_{1,2} & & & \\ [k]_{2,1} & [k]_{2,2} & [k]_{2,3} & & \\ & [k]_{C,C-1} & [k]_{C,C} & [k]_{C,C-I} & \\ & & & [k]_{II,III} & [k]_{II,II} & [k]_{II,I} \\ & & & [k]_{I,II} & [k]_{I,I} & \end{bmatrix} \begin{bmatrix} [\delta]_1 \\ [\delta]_2 \\ [\delta]_C \\ [\delta]_{II} \\ [\delta]_I \end{bmatrix} \quad (3.27)$$

Proceeding as before, one can write the recursion relations Equations 3.18 through 3.21. The index j stands for the j^{th} substructure counted from either end. When $j = C-1$ (where C is the index of the central substructure), Equation 3.21 becomes:

$$[\delta]_{C-1} = - [k]_{C-1,C-1}^{*-1} \left[[k]_{C-1,C} [\delta]_C - [F]_C^* \right] \quad (3.28)$$

where:

$$[F]_C^* = [F]_C - [k]_{C,C-1} [k]_{C-1,C-1}^{*-1} [F]_{C-1} \quad (3.29)$$

Similarly, when $j = C-I$, Equation 3.21 becomes:

$$[\delta]_{C-I} = - [k]_{C-I,C-I}^{** -1} \left[[k]_{C-I,C} [\delta]_C - [F]_C^{**} \right] \quad (3.30)$$

where:

$$[F]_C^{**} = [F]_C - [k]_{C,C-I} [k]_{C-I,C-I}^{** -1} [F]_{C-I} \quad (3.31)$$

Double stars on the matrices signify that the recursion relation is carried out from the front end, with Roman numerals.

The matrix equation for the central substructure may now be written from Equation 3.2 as:

$$[F]_C = [k]_{C,C-1} [\delta]_{C-1} + [k]_{C,C} [\delta]_C + [k]_{C,C-I} [\delta]_{C-I} \quad (3.32)$$

Substituting Equations 3.28 through 3.31 in Equation 3.32:

$$[F]_C = [k]_{C,C} [\delta]_C \quad (3.33)$$

where:

$$[k]_{C,C} = [k]_{C,C} - [k]_{C,C-1} [k]_{C-1,C-1}^{-1} [k]_{C-1,C} - [k]_{C,C-I} [k]_{C-I,C-I}^{-1} [k]_{C-I,C} \quad (3.34)$$

and

$$[F]_C = [F]_C - [k]_{C,C-1} [k]_{C-1,C-1}^{-1} [F]_{C-1} - [k]_{C,C-I} [k]_{C-I,C-I}^{-1} [F]_{C-I} \quad (3.35)$$

After the deflections $[\delta]_C$ are obtained from equation 3.33, the deflections for the remaining structures are obtained by successive applications of the recursion relations of Equations 3.18 to 3.21.

The recursion process requires the inversion of matrices whose largest order is equal to the order of the largest matrix $[k]_{j,j}$. This enables the programmer to conserve the computer core storage, provided he uses the auxiliary storage units for the large blocks of information pertaining to individual substructures. Hence, a large structure may now be analyzed. When auxiliary storage units are used in the program, the benefits derived from a large capacity are partially offset by an increase in computer time. This loss of efficiency is due to the fact that access time is typically much greater for auxiliary storage than for the core

storage. It may, however, be noted that the time of solution varies as a linear function of the number of substructures in the structure, rather than as the cube of the number of unknown deflections $\{\delta\}$, as is the case in ordinary techniques for solving equations.

3.3 VERIFICATION OF COMPUTER CODE

A computer program is written to analyze a structure composed of plate, beam and bar elements. The program utilizes the series method of substructures described in the previous section. It is used for the analysis of the bridge structures which are discussed later.

The logic of the program is verified with the cantilever beam problem presented in the second chapter. The cantilever beam problem was solved several times. Each time the beam was divided into different substructures and the scheme of numbering the node points was changed. The deflections of the beam were found identical in each case. For each element nodal forces were computed from the known deflections. The forces acting at each node were then superimposed for equilibrium checks of the nodal forces.

The cantilever beam lies entirely in one plane and is made up only of plate elements. Hence, it does not check certain aspects of the program. Therefore, the program is also verified with a bridge supported on two longitudinal girders. The computed deflections of all the nodes of the bridge were identical for the various combinations of substructures (Figure 5). The equilibrium conditions were also checked at each node of

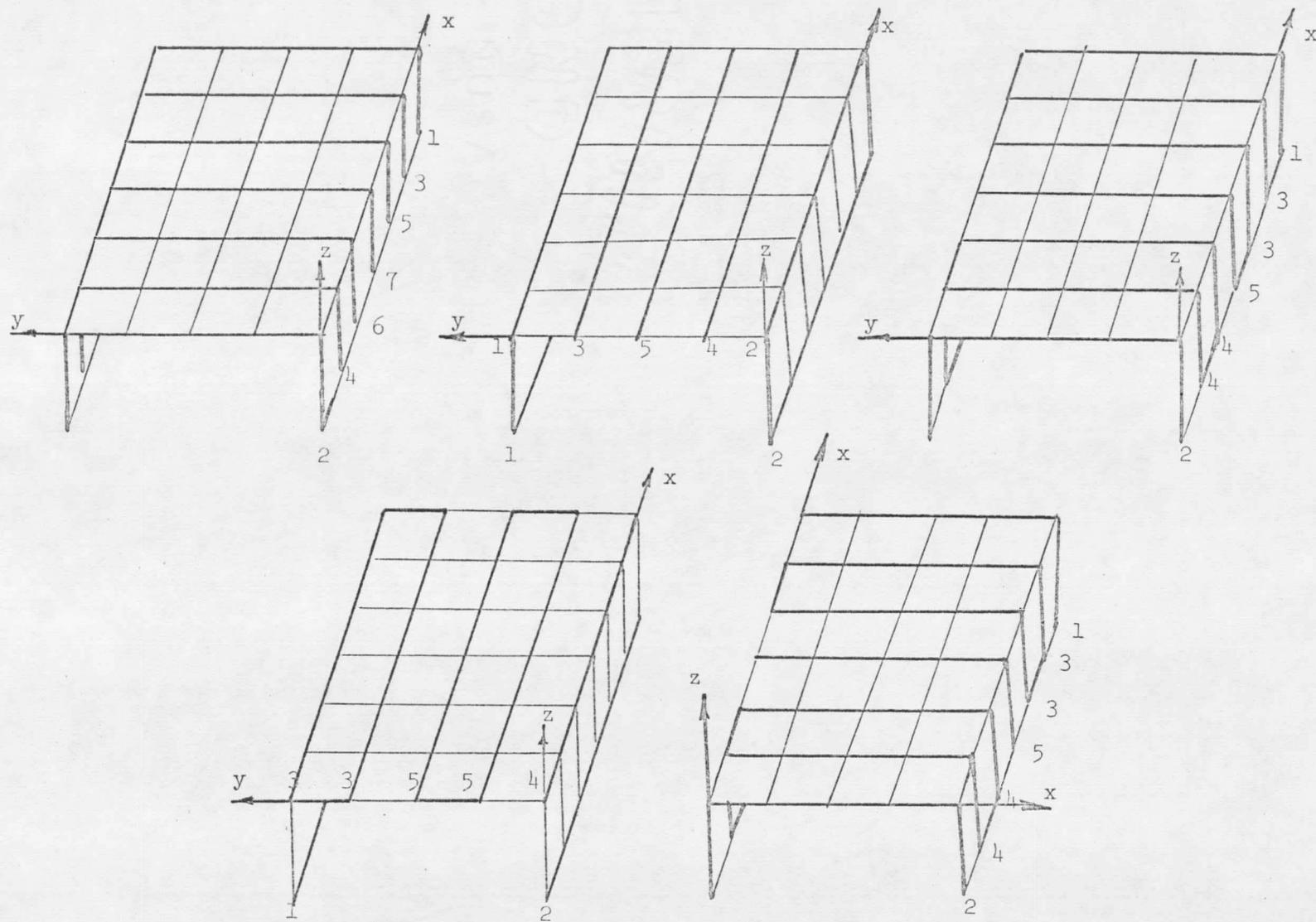


Figure 5. Structure divided into different substructures.

the bridge. The bridge was also solved with axial diaphragms at its supports. The deflections of the various nodes due to applied loads were identical when the structure was subdivided into two different substructures.

3.4 COMPARISONS WITH EXISTING SOLUTIONS

The applications of the finite element method of analysis are varied. The method was recently applied to composite I-beam bridges (46). The literature on the finite element method is full of evaluations of the method in comparison to other methods of solution. The evaluations, which are based on comparisons with exact solutions for simple problems and with the experimental results for the more complex problems, have proved a great success of the method. Therefore, in the present study of the bridge behavior under the action of horizontal loads, no attempt is made to justify the use of the method. However, certain comparisons are made to investigate the validity of the method and to illustrate the differences in solutions resulting from the various assumptions made in formulating the analytical models and other approximate methods of solution.

Comparisons are made for vertical loads on three composite I-beam bridges of different span lengths and girder sizes. All the bridges are solved by the finite element method by Gustafson (46). In his model, plate elements have only five degrees of freedom U , V , W , θ_x , and θ_y ; and the longitudinal and transverse girders are treated as beam elements. The stiffness of the beam elements is computed by assuming the nodes of the beam elements to coincide with the neutral surface of the slab.

The first bridge studied is a five girder bridge supported over a span of sixty feet (Figure 6). The bridge was solved by Vitols, Clifton and Au by the orthotropic plate theory in order to compare their more accurate treatment of the eccentric stiffeners with the conventional technique (19).

In the present analysis the slab of the bridge is represented by a mesh of sixteen longitudinal by ten transverse rectangular plate elements of equal size. Each element of the slab has six degrees of freedom; U , V , W , θ_x , θ_y , and θ_z . The webs of the longitudinal girders are treated as plate elements and their flanges are treated as beams lying in horizontal planes.

Table I gives a comparison of the present investigation for the midspan loading on the girders with the orthotropic plate analysis of Vitols, Clifton and Au, and with the finite element analysis of Gustafson. It also presents moment coefficients for the loads at the quarter span and at three-quarter span of the bridge. The data in Table I shows a somewhat different distribution of load compared to the orthotropic plate theory and Gustafson's finite element model.

It is observed that the differences in the midspan moments between the orthotropic plate analysis and Gustafson's model are greatest when the exterior girder A is loaded. They reduce when girder B is loaded and are a minimum when girder C is loaded. In the present analysis, the moments carried by the interior girders B and C, when the vertical loads are

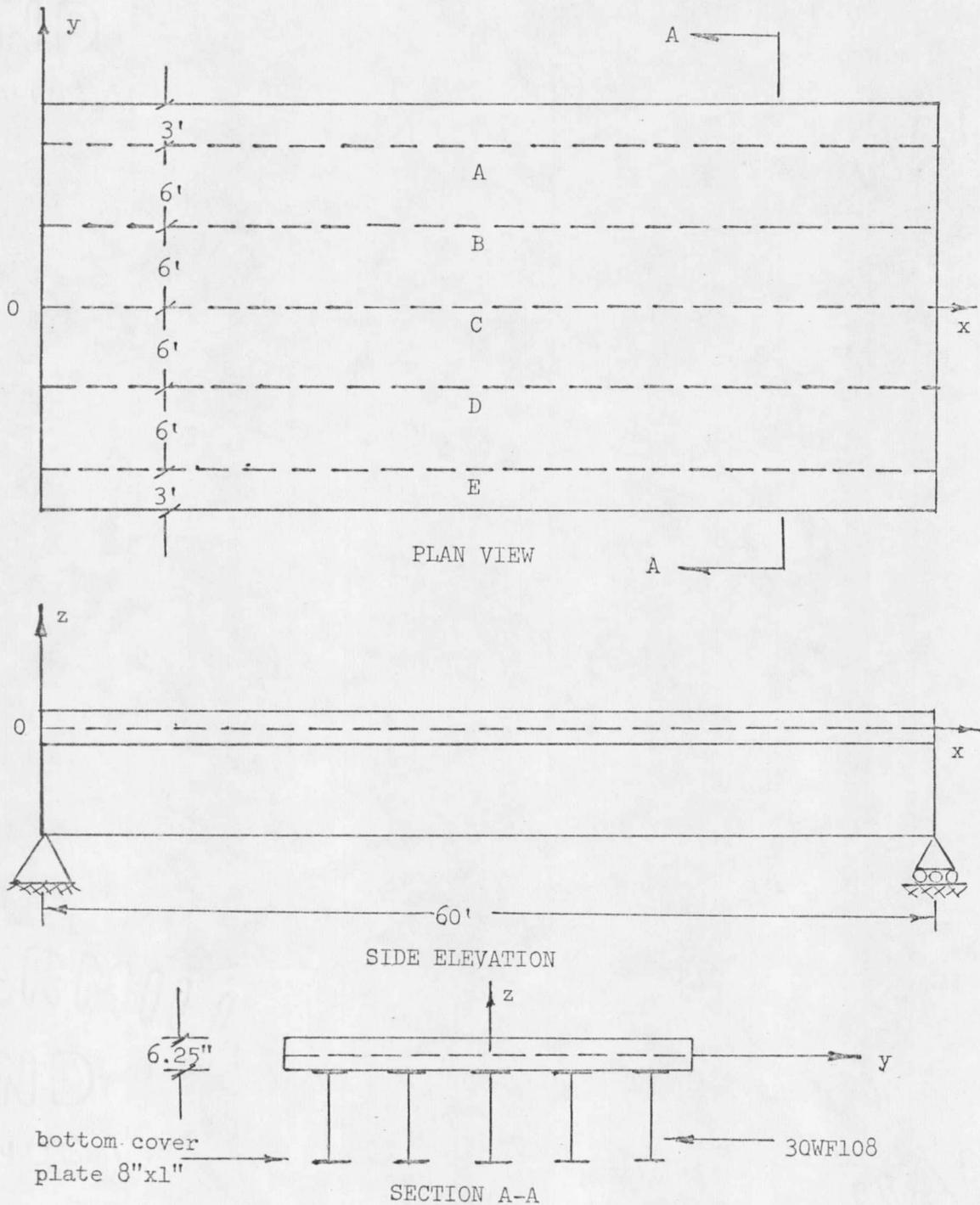


Figure 6. Details of simply supported five girder bridge.

TABLE I. Comparison of Moment Coefficients C_m for 60 foot Span Composite I-Beam Bridge.

Load Position	Basis of Comparison *	Coefficient of Moment $C_m^{\#}$ Mid-Span Section of Beam				
		A	B	C	D	E
Mid-Span A	1	0.1754	0.0872	0.0089	-0.0128	-0.0085
	2	0.1961	0.0592	0.0085	-0.0058	-0.0066
	3	0.1818	0.0651	0.0191	-0.0019	-0.0141
Mid-Span B	1	0.0543	0.1324	0.0540	0.0139	-0.0046
	2	0.0598	0.1255	0.0536	0.0163	-0.0051
	3	0.0636	0.1194	0.0510	0.0169	-0.0010
Mid-Span C	1	0.0097	0.0539	0.1228	0.0539	0.0097
	2	0.0090	0.0540	0.1238	0.0540	0.0090
	3	0.0185	0.0512	0.1105	0.0512	0.0185
1/4-Span A	3	0.0759	0.0447	0.0171	-0.0005	-0.0123
1/4-Span B	3	0.0436	0.0377	0.0305	0.0129	0.0002
1/4-Span C	3	0.0165	0.0306	0.0307	0.0306	0.0165
3/4-Span A	3	0.0795	0.0431	0.0133	-0.0018	-0.0092
3/4-Span B	3	0.0420	0.0388	0.0325	0.0129	-0.0012

cont.

TABLE I. - Cont.

Load Position	Basis of Comparison *	Coefficient of Moment C_m for Quarter-Span Section of Beam				
		A	B	C	D	E
Mid-Span A	1	0.0709	0.0625	0.0067	-0.0091	-0.0060
	2	0.0875	0.0400	0.0066	-0.0042	0.0047
	3(a)	0.0647	0.0507	0.0271	0.0030	-0.0205
	3(b)	0.0817	0.0421	0.0112	-0.0025	-0.0074
Mid-Span B	1	0.0364	0.0463	0.0354	0.0102	-0.0033
	2	0.0405	0.0412	0.0350	0.0122	-0.0037
	3(a)	0.0502	0.0303	0.0266	0.0149	0.0029
	3(b)	0.0408	0.0401	0.0333	0.0126	-0.0018
Mid-Span C	1	0.0072	0.0353	0.0400	0.0353	0.0072
	2	0.0070	0.0352	0.0408	0.0352	0.0070
	3(a)	0.0267	0.0266	0.0184	0.0266	0.0267
	3(b)	0.0108	0.0334	0.0367	0.0334	0.0108
1/4-Span A	3(a)	0.1311	0.0505	0.0224	0.0020	-0.0185
	3(b)	0.0351	0.0243	0.0099	-0.0005	-0.0063
1/4-Span B	3(a)	0.0498	0.0920	0.0317	0.0119	0.0021
	3(b)	0.0235	0.0152	0.0152	0.0087	-0.0001
1/4-Span C	3(a)	0.0222	0.0317	0.0798	0.0317	0.0222
	3(b)	0.0096	0.0152	0.0128	0.0152	0.0096
3/4-Span A	3(a)	0.0267	0.0283	0.0178	0.0025	-0.0128
	3(b)	0.1502	0.0393	0.0056	-0.0027	-0.0048
3/4-Span B	3(a)	0.0281	0.0107	0.0119	0.0096	0.0022
	3(b)	0.0378	0.1063	0.0375	0.0078	-0.0020

Moment = C_m x Load x Span

- * 1: Orthotropic plate theory (19)
 2: Gustafson's model (46).
 3: Present investigation
 3(a): Moment coefficients at quarter-span
 3(b): Moment coefficient at three-quarter-span

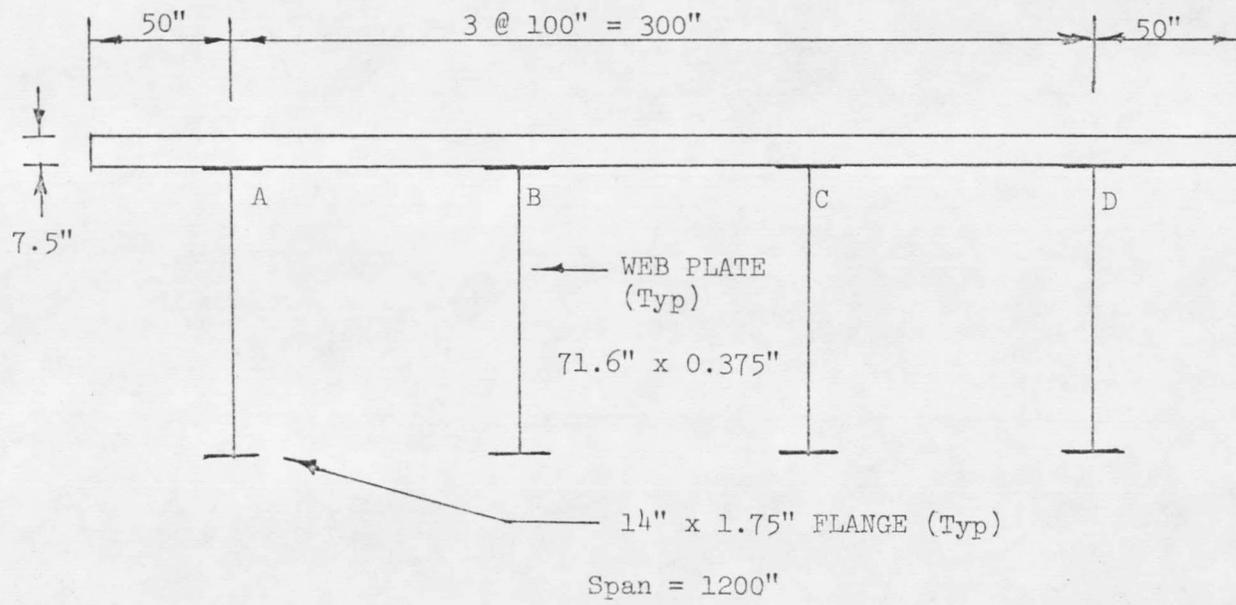
directly applied on these girders, are lower than the moments predicted by the orthotropic plate theory or Gustafson's finite element model. When the exterior girder A is loaded, the moment at the midspan of girder A is a little larger than the moment predicted by the orthotropic plate theory but is smaller than the moment given by Gustafson's model. This distribution of moments to the unloaded girders is similar in all the cases. The distribution of the load to the unloaded girders is a maximum when the central girder C is loaded, it is less when girder B is loaded and is a minimum when the exterior girder A is loaded. It is also observed that the moment coefficients at the quarter span of the loaded girders, computed by the present technique, are smaller than the corresponding coefficients predicted by Gustafson's model or the orthotropic plate theory. In Table I, the moment coefficients are also listed for three-quarter span section, with the quarter span moment coefficients. It is seen from these coefficients that the distribution of the moment along the span is not symmetric about the midsection of the bridge. The distribution of the moment coefficients is more uniform at the quarter spans than at the midspan when the loads are applied at the midsection of the various longitudinal girders.

The differences between the finite element model of Gustafson and the present model may be attributed to the lack of three dimensional nature of the Gustafson's model. Gustafson has treated the longitudinal girders as an assemblage of beam elements whose nodes coincide with the middle plane of the slab. In other words, the longitudinal girders are

assumed to stiffen the edges of the slab elements, and the entire bridge lies in one horizontal plane. Hence, when the vertical loads are applied, the model will not have reactions in the horizontal plane. The three dimensional model is found to have rather significant transverse and longitudinal reactions (Table VII). Similarly, the space-type behavior of the bridge leads to a different distribution of the vertical reactions at the two ends of the bridge. Since the horizontal reactions occur at the bottom plane, they will affect the moments. The moments in the various girders will be modified roughly in proportion to the magnitude and the elevation of the longitudinal reactions.

The differences with the orthotropic plate are attributed to two reasons: (1) the entire plate is in one plane as is the case with Gustafson's model, (2) the orthotropic plate model is obtained by smearing stiffness of the girders to obtain an equivalent plate model. It may be noted that the flexural and torsional rigidities of the equivalent orthotropic plates are obtained in an approximate manner, and the method is justifiable only if the longitudinal stiffeners are spaced so that the ratios of the stiffener spacing to the width of the bridge cross section are very small. Besides, the orthotropic plate theory ignores the stresses associated with the warping of the cross section which may be induced due to nonuniform distribution of the vertical loads.

The second bridge compared is a four-girder composite I-beam bridge (Figure 7). The bridge is 400 inches wide and is simply



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Figure 7. Details of 100' span composite I-beam bridge.

supported over a span of 1200 inches. The web plates of the longitudinal girders are 71.6" x 0.375" and the top and bottom flanges of the girders are 14.0" x 1.75". The bridge is analyzed for a line load of magnitude $\sin \frac{\pi x}{L}$; along an exterior girder A for one loading and along an interior girder B for the other loading.

The equivalent nodal forces for the sine loads are obtained by assuming that the deflected shape of the edge of the loaded girder can be represented by third order polynomials between the nodes of the elements. Alternately, it is assumed that the slab and the girder plate elements are compatible along their common edge, and the transverse displacement of the edge between two nodes is described by a third order polynomial.

The results of the analysis are compared in Table II with the results of Wright* and Gustafson. Wright's results are based on the direct stiffness method following the general assumptions described in Vlasov (47) for thin walled beam analysis. The structure is assumed to be an assemblage of plate elements which run the entire length of the structure from support to support.

The transverse slab moments M_x per unit length of the slab and the longitudinal forces N_x per unit width of the slab compare extremely well between Gustafson's finite element model and Wright's thin beam approach. However, these values are consistently higher in comparison to the present investigation, i.e., the present model results in smaller stresses in the slab.

* The results are obtained from Gustafson's thesis (46).

TABLE II. Comparison of Internal Forces for a Composite I-Beam Bridge (Figure 7).

1. Sine load acting on external girder A

Force	Location x=	Basis of Comparison	Transverse Location			
			A	B	C	D
M_x in lbs/in Transverse slab moment per unit length (tension at bottom is +ve)	0.50L	Wright*	2.10	-17.37	-9.70	-0.56
		Gustafson	2.06	-16.76	-9.14	-0.52
		Present	1.69	-12.37	-5.07	-0.51
	0.25L	Wright	1.48	-12.28	-6.89	-0.39
		Gustafson	1.46	-11.86	-6.49	-0.37
		Present	0.83	-7.07	-2.59	0.55
N_x = lbs/in force in slab per unit width (tension + ve)	0.50L	Wright	-13.11	-7.49	-2.21	2.78
		Gustafson	-13.63	-7.87	-2.37	2.84
		Present	-10.17	-5.69	-1.58	-2.20
	0.25L	Wright	-9.28	-5.31	-1.56	1.96
		Gustafson	-9.66	-5.54	-1.64	1.98
		Present	-7.26	-3.96	-1.05	1.57

2. Sine load acting on internal girder B

Force	Location x=	Basis of Comparison	Transverse Location			
			A	B	C	D
M_x Transverse slab moment per unit length	0.50L	Wright	-0.64	27.25	-0.14	-0.90
		Gustafson	-0.61	26.92	-0.30	-0.85
		Present	-0.51	21.40	3.34	-0.61
	0.25L	Wright	-0.45	19.22	-0.10	-0.64
		Gustafson	-0.43	19.06	-0.20	-0.60
		Present	0.01	12.48	-2.34	-0.25
N_x Force in slab per unit width	0.50L	Wright	-7.54	-6.11	-4.23	-2.18
		Gustafson	-7.82	-6.39	-4.45	-2.29
		Present	-5.64	-4.73	-3.24	-1.32
	0.25L	Wright	-5.33	-4.32	-2.99	-1.54
		Gustafson	-5.49	-4.56	-3.19	-1.58
		Present	-3.92	-3.44	-2.32	-1.02

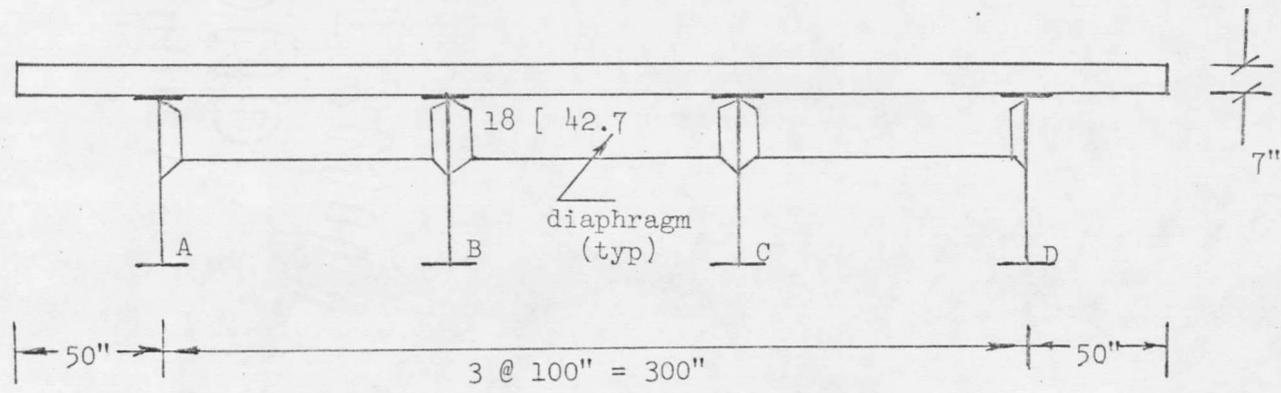
* The results are obtained from Gustafson's thesis.

In Wright's approach, any load between the simple supports of the bridge has to be represented by a series of sine loads acting along the edges of the plate elements. If the supports were not simple supports then the application of the method to the bridge problem would be extremely cumbersome. The ease of the finite element approach in treating various boundary conditions and loads is the prime advantage of the method.

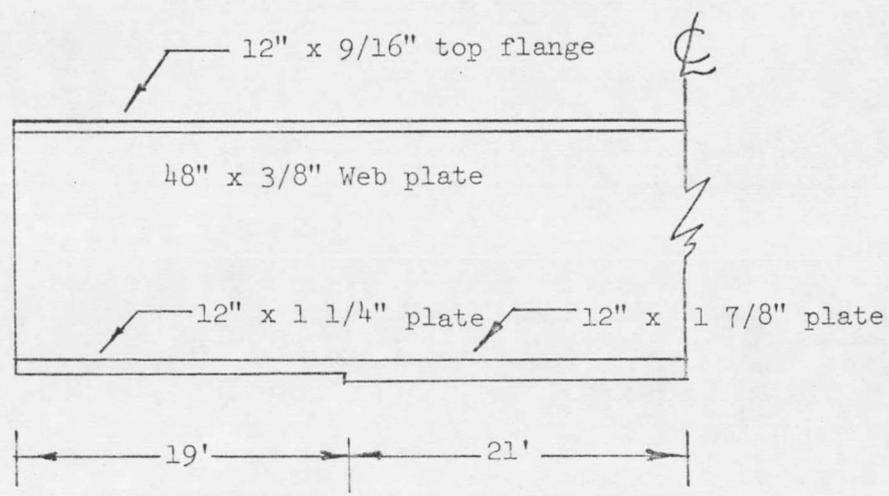
A final comparison is made with the eighty foot span, four girder bridge of Figure 8. The bridge is analyzed without diaphragms in one case and with diaphragms spaced at an interval of quarter-span and at the end sections in the other case.

The results of the analysis are presented in Tables III, IV, and V along with Gustafson's results. The girder reactions R_z for the case when the diaphragms are provided are read from Gustafson's plots. Figures 9 and 10 are plotted to indicate the influence of the diaphragms in distributing vertical loads to various unloaded girders.

The influence line coefficients C_m , for midspan girder moments in the present analysis has a distribution similar to Gustafson's solution. Here, it may be noted that the present analysis gives larger moments for the loaded girders than the finite element analysis of Gustafson. This distribution is in contradiction to the observation made during the comparison of five girder bridge of Figure 6. Here, the present method of analysis has resulted in smaller moment coefficients than Gustafson's computations (Table V).



Bridge Cross Section



Typical Girder Elevation

Figure 8. 80' span, four girder bridge details.

TABLE III. Influence Coefficient C_m for Mid-span Girder Moments for an 80' Span, Four Girder Bridge.

Mid-span transverse load location	Girder	Without Diaphragms		With Diaphragms	
		Gustafson	Present Analysis	Gustafson*	Present Analysis
A	A	.1944	.1996	.1828	.1866
	B	.0621	.0600	.0754	.0763
	C	.0077	.0068	.0154	.0135
	D	-.0166	-.0165	-.0262	-.0264
B	A	.0621	.0604	.0763	.0761
	B	.1243	.1311	.1027	.1041
	C	.0533	.0511	.0533	.0564
	D	.0079	.0075	.0156	.0134

* Diaphragms are treated as beam elements with their nodes at the middle plane of the slab.

TABLE IV. Influence Coefficients for Girder Reactions R_z for an 80' Span
Four Girder Bridge with Midspan Loadings

Girder	Transverse Load Location	Gustafson		Present Analysis			
		No Diaph.	With Diaph.	No Diaphragms X=0* X=L**		With Diaphragms X=0 X=L	
A	A	.3764	.3380	.4634	.4015	.3930	.3614
	B	.1582	.1940	.0910	.1569	.1663	.1967
	C	.0082	.0450	-.0400	.0015	.0105	.0374
	D	-.0401	-.0120	-.0073	-.0560	-.0696	-.0945
B	A	.1513	.1890	.0698	.1372	.1483	.1788
	B	.1711	.1360	.2792	.1865	.1791	.1427
	C	.1576	.1280	.1699	.1551	.1441	.1233
	D	.0144	.0460	-.0260	.0173	.0283	.0543

* U, V, W are prevented (pinned)

** V, W are prevented (roller)

TABLE V. Influence Coefficient for the Vertical Reactions for an 80' Span Four Girder Bridge with Quarter Span Loadings

Gir- der	Trans- verse Load Loca- tion	Gustafson No Diaph.*	Present Analysis							
			Load at x = 19'				Load at x = 61'			
			No Diaph.		With Diaph.		No Diaph.		With Diaph.	
			x=0 [#]	x=L ^{\$}	x=0	x=L	x=0	x=L	x=0	x=L
A	A	.6335	.7464	.1757	.6734	.1633	.2111	.6778	.1740	.6197
	B	.1513	.0595	.0921	.1557	.1018	.0588	.1445	.0937	.2168
	C	-.0062	-.0449	.0123	-.0263	.0270	-.0178	-.0176	.0173	.0080
	D	-.0288	.0079	-.0406	-.0405	-.0541	-.0106	-.0380	-.0473	-.0815
B	A	.1461	.0428	.0803	.1406	.0915	.0457	.1281	.0828	.2009
	B	.4208	.6000	.0641	.4260	.0597	.1022	.4556	.0682	.3361
	C	.1760	.1479	.0689	.2071	.0489	.0941	.1800	.0584	.2016
	D	-.0016	-.0346	.0220	-.0011	.0368	-.0087	-.0053	.0279	.0234

* Load at quarter span

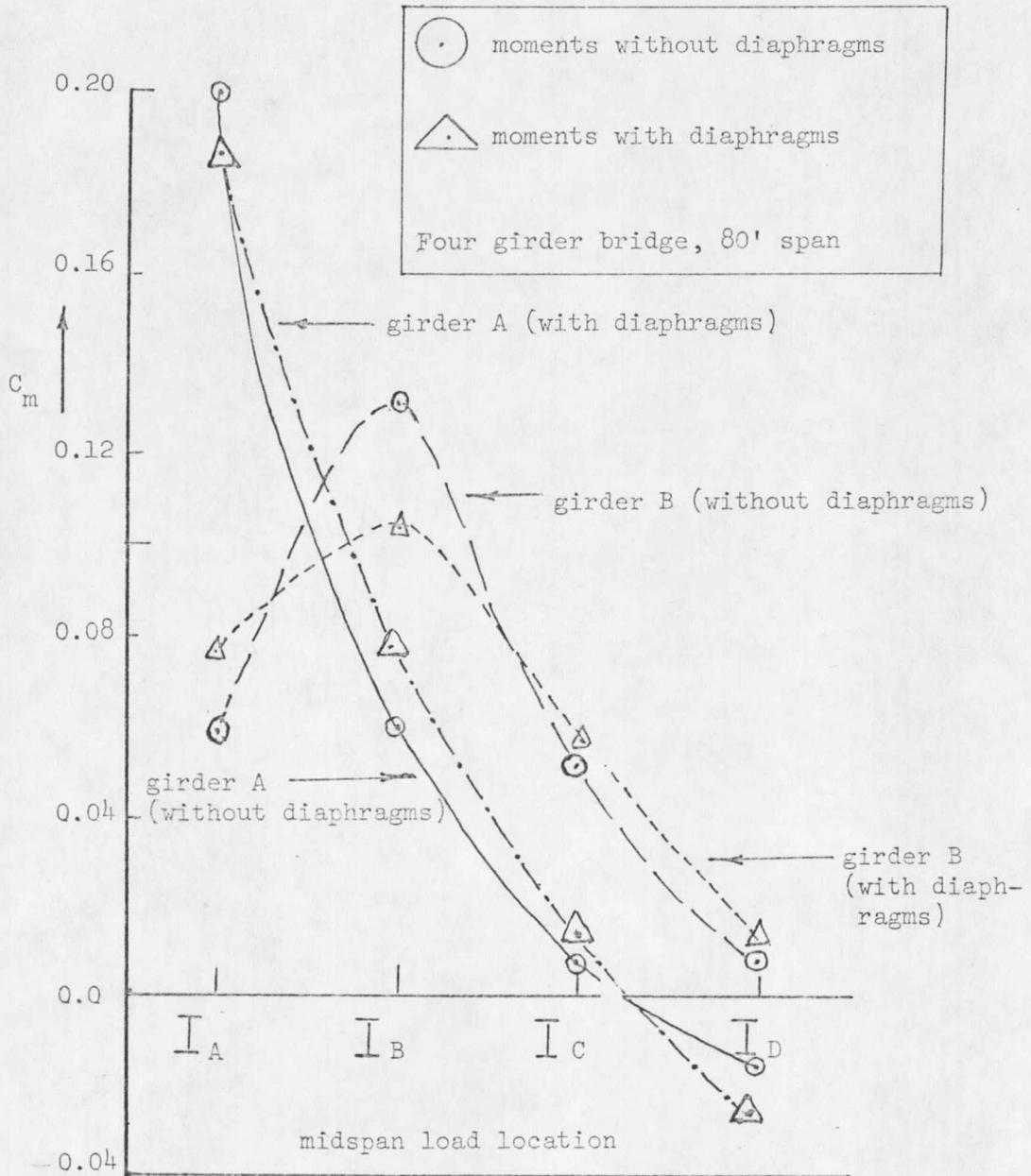
pinned end

\$ roller end

It is interesting to note that the vertical reactions R_z are not equal at both ends of the girders when the load is applied at the midspan. Similarly, we observe that the reactions R_z at the pinned support at $x=0$ is not equal to the reactions R_z at the roller support at $x=L$ when the loads are applied at corresponding points. Obviously, the phenomenon is the result of the nonsymmetric boundary conditions of the bridge and the depth of the bridge girders which is properly considered in this analysis.

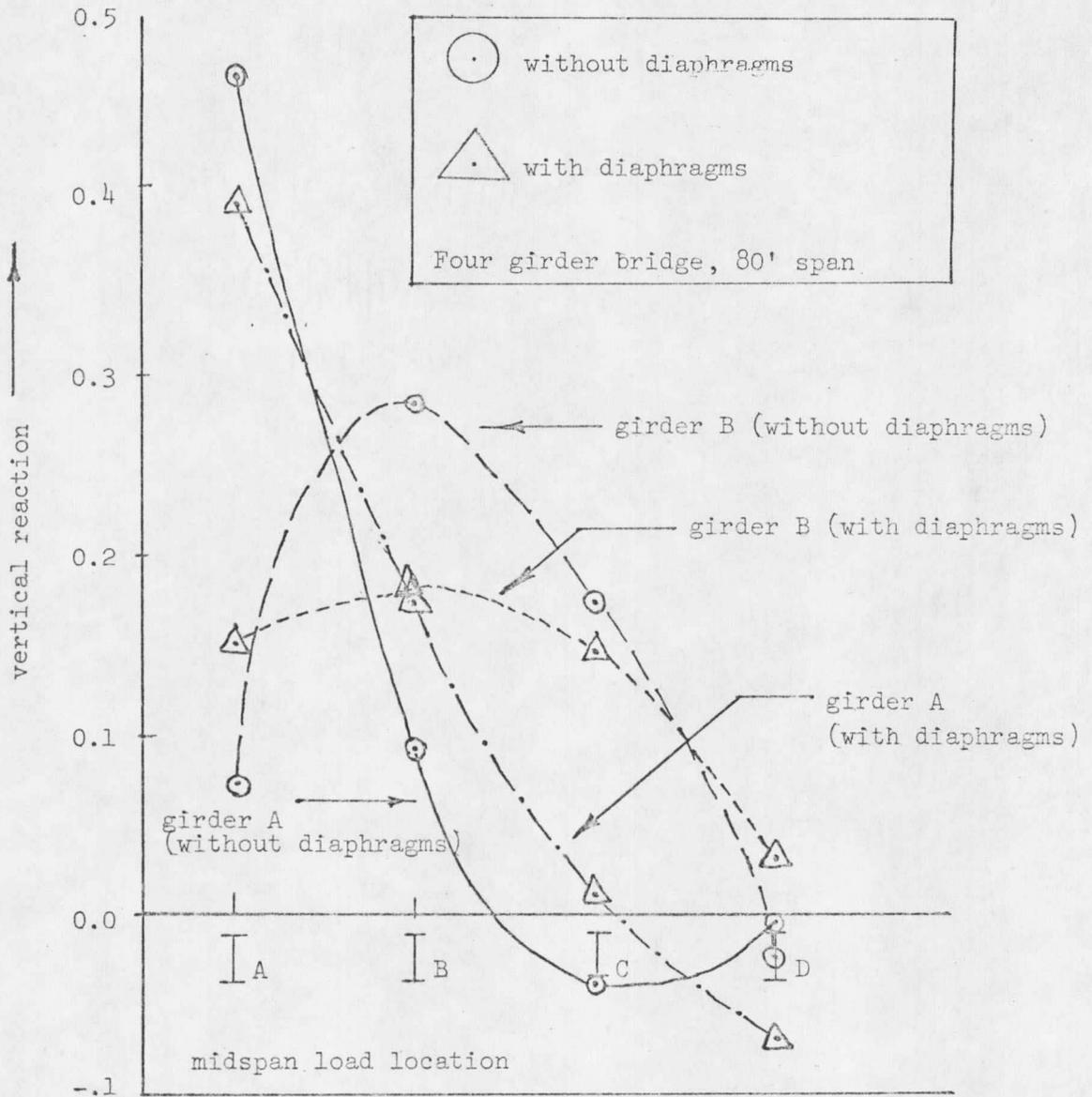
From Figures 9 and 10, considerable redistribution of the load is noted when the interior girder B is loaded. Whereas, when the exterior girder A is loaded, the distribution of load to the unloaded girders is skimpy. Inclusion of diaphragms in the bridge relieves the loaded girders. The change in the load carried by the loaded interior girder B is larger than the loaded exterior girder A.

From Figure 10, it may be observed that due to the presence of the diaphragms the reactions R_z at the pinned end of the interior girder B is almost the same for the load on the exterior girder A and for the load on the interior girders B and C. Similarly, in Table IV the reactions R_z at the roller end ($x=L$) of the girder B is seen to be greater when the load is applied on the exterior girder A than when the load is applied on the interior girder B itself. This clearly is due to greater redistribution of the load to the unloaded exterior girders, which occurs in the presence of the diaphragms.



Influence lines for longitudinal moments of midspan of girders A and B.

Figure 9. Effect of diaphragms on composite girder moments.



Influence lines for vertical reactions at $x=0$ of girders A and B.

Figure 10. Effect of diaphragms on girder reactions.

In the above comparisons the bridges were simply supported. Alternately, all the girders were supported on pins at one end which prevented U, V, and W displacements of the bottom flange at this end. The other end of the girders was supported on the rollers which permitted extension U in the longitudinal direction. The longitudinal translations of the girders may be prevented partially due to friction between the girders and the rollers or due to rust on the rollers. This then would alter the boundary conditions for the bridge and hence may alter the load distribution characteristics of the bridge.

Tables VI and VII present data for the simply supported bridge (longitudinal extensions permitted at one end) and the bridge pinned at both ends. As may be evidenced from Table VI, the moment coefficients C_m at midspan and at quarterspan of the bridge, are considerably smaller for the case when the two ends of the bridge are pinned than when one end is pinned and the other is on rollers. The magnitudes of moments at midspan and at quarterspans of the bridge pinned at both ends, are similar to a beam fixed at both ends.

Table VII shows a little variation in the vertical reactions R_z due to the differences in the boundary conditions of the bridge. But radical changes in longitudinal reactions R_x are noted. In both cases, the transverse reactions R_y are negligible quantities.

Because of the substantial changes that occur in the load distribution characteristics of the bridge in these two cases, it is

TABLE VI. Effect of Boundary Conditions on Moment Coefficients C_m

Load Position	Boundary Conditions	Coefficient of Moment C_m for Mid Span Section of Beam				
		A	B	C	D	E
Mid Span Beam A	Non Sym.*	0.1818	0.0651	0.0191	-0.0019	-0.0141
	Sym.	0.1040	0.0172	-0.0024	-0.0028	0.0018
Mid Span Beam B	Non Sym.	0.0636	0.1194	0.0510	0.0169	-0.0010
	Sym.	0.0162	0.0801	0.0225	0.0020	-0.0018
Mid Span Beam C	Non Sym.	0.0185	0.0512	0.1105	0.0512	0.0185
	Sym.	-0.0027	0.0227	0.0791	0.0227	-0.0027
Qtr. Span Beam A	Non Sym.	0.0759	0.0447	0.0171	-0.0005	-0.0123
	Sym.	0.0189	0.0079	-0.0008	0.0017	0.0012
Qtr. Span Beam B	Non Sym.	0.0436	0.0377	0.0305	0.0129	0.0002
	Sym.	0.0072	0.0087	0.0099	0.0018	-0.0010
Qtr. Span Beam C	Non Sym.	0.0165	0.0306	0.0307	0.0306	0.0165
	Sym.	-0.0011	0.0100	0.0089	0.0100	-0.0011

		C_m for Quarter Span Section of Beam				
		A	B	C	D	E
Mid Span Beam A	Non Sym.	0.0647	0.0507	0.0271	0.0030	-0.0205
	Sym.	-0.0070	-0.0005	-0.0010	-0.0003	0.0016
Mid Span Beam B	Non Sym.	0.0502	0.0303	0.0266	0.0149	0.0029
	Sym.	-0.0007	-0.0067	0.0010	-0.0002	-0.0003
Mid Span Beam C	Non Sym.	0.0267	0.0266	0.0184	0.0266	0.0267
	Sym.	-0.0011	0.0011	-0.0060	0.0011	-0.0011
Qtr. Span Beam A	Non Sym.	0.1311	0.0505	0.0224	0.0020	-0.0185
	Sym.	0.0765	0.0124	0.0018	-0.0004	-0.0023
Qtr. Span Beam B	Non Sym.	0.0498	0.0920	0.0317	0.0119	0.0021
	Sym.	0.0120	0.0637	0.0123	0.0011	0.0000
Qtr. Span Beam C	Non Sym.	0.0220	0.0317	0.0798	0.0317	0.0222
	Sym.	0.0019	0.0124	0.0607	0.0124	0.0019

* Sym. U, V, W are fixed at both ends of the girders.

Non Sym. U, V, W are fixed at near end of the girders, and V, W are fixed at far end of the girders.

TABLE VII. Effect of Boundary Conditions on Support Reactions R_x , R_y , and R_z .

Load on Beam	At $x=$	B.C.	Reactions of Beam	$x = 0$			$x = L$		
				R_x	R_y	R_z	R_x	R_y	R_z
A	L/2	NS*		939.9	-5.9	449.2		6.0	354.2
		S		2570.6	-0.2	417.6	-2570.6	0.0	417.6
B	L/2	NS		-567.1	-2.6	116.2		2.6	172.7
		S		731.4	0.2	122.0	-731.4	0.2	122.0
C	L/2	NS		-834.4	0.1	-39.8		-0.5	33.7
		S	A	-34.7	0.0	-16.5	34.7	0.0	-16.5
A	L/4	NS		1022.7	-4.1	721.0		4.6	155.8
		S		2307.4	0.2	704.8	-1676.9	0.1	180.6
B	L/4	NS		-762.7	-3.1	86.5		2.5	99.0
		S		179.8	-1.0	87.1	-756.7	0.5	83.3
C	L/4	NS		-779.4	0.0	-53.6		-0.1	35.5
		S		-238.5	0.0	-44.2	-186.2	-0.1	12.8
A	L/2	NS		-522.4	-5.8	110.7	0.0	6.0	167.7
		S		784.9	0.0	126.9	-784.9	0.0	126.9
B	L/2	NS		665.0	-2.8	232.5	0.0	2.7	154.4
		S		1534.2	0.0	229.9	-1534.2	0.0	229.9
C	L/2	NS		315.3	0.2	165.3	0.0	-0.3	143.9
		S	B	864.2	0.1	154.3	-864.2	0.1	154.3
A	L/4	NS		-718.5	-4.6	85.9	0.0	4.7	96.3
		S		235.1	-0.2	94.4	-788.4	0.3	86.6
B	L/4	NS		1105.6	-2.3	532.5	0.0	2.4	61.9
		S		1780.6	-0.2	532.5	-871.7	0.3	82.8
C	L/4	NS		92.2	-0.4	167.7	0.0	0.0	62.3
		S		532.1	-0.4	163.5	-603.0	0.2	72.5
A	L/2	NS		-823.3	-5.7	-35.7	0.0	5.7	37.1
		S		-11.6	0.1	-11.7	-11.6	0.1	-11.7
B	L/2	NS		299.5	-3.0	160.4	0.0	2.9	140.5
		S		852.7	-0.1	149.8	-852.7	-0.1	149.8
C	L/2	NS		1038.2	0.0	249.1	0.0	0.0	144.7
		S	C	1482.2	0.0	224.4	-1482.2	0.0	224.4
A	L/4	NS		-770.1	-5.0	-50.9	0.0	4.8	37.2
		S		-217.6	-0.7	-40.7	-197.7	0.5	15.2
B	L/4	NS		81.0	-2.0	163.5	0.0	2.3	61.0
		S		523.5	0.2	159.4	-596.9	0.2	70.5
C	L/4	NS		137.4	0.0	52.2	0.0	0.0	54.3
		S		1771.1	0.0	511.5	-779.9	0.0	793.3

TABLE VII Cont.

Load on Beam	at $x=$	B.C.	Reactions of Beam	$x = 0$			$x = L$		
				R_x	R_y	R_z	R_x	R_y	R_z
A	L/2	NS	D	-260.6	-5.6	-34.2	0.0	5.5	-17.4
		S		-137.9	0.1	-27.6	137.9	0.1	-27.6
B	L/2	NS	D	-193.4	-2.8	28.3	0.0	3.0	53.3
		S		132.2	0.0	22.8	-132.2	0.0	22.8
C	L/2	NS	D	315.3	0.2	165.3	0.0	0.3	143.9
		S		864.2	-0.1	154.3	-864.2	-0.1	154.3
A	L/4	NS	D	-161.4	-4.9	-20.8	0.0	4.7	-5.4
		S		-91.0	-0.6	-19.2	79.1	0.5	-14.3
B	L/4	NS	D	-306.5	-2.4	-10.5	0.0	2.5	36.1
		S		-59.9	-0.2	-13.4	-197.2	0.3	26.2
C	L/4	NS	D	92.2	0.4	167.7	0.0	0.0	62.3
		S		532.1	0.4	163.5	-603.0	0.2	72.5
A	L/2	NS	E	666.5	-5.5	9.9	0.0	5.4	-41.5
		S		-32.6	0.1	-5.2	32.6	0.1	-5.2
B	L/2	NS	E	-206.0	-2.8	-37.5	0.0	2.9	-20.9
		S		-103.1	0.0	-24.5	103.1	0.0	-24.5
C	L/2	NS	E	-834.4	-0.1	-39.8	0.0	0.5	33.7
		S		-34.7	0.0	-16.5	34.7	0.0	-16.5
A	L/4	NS	E	627.2	-4.8	14.8	0.0	4.6	-33.8
		S		151.9	-0.6	10.8	198.1	0.5	-18.2
B	L/4	NS	E	-117.5	-2.4	-22.0	0.0	2.5	-8.0
		S		-60.7	-0.3	-15.6	59.2	0.3	-12.8
C	L/4	NS	E	-779.4	0.0	-53.6	0.0	0.1	35.5
		S		-238.5	0.0	-44.2	-186.2	-0.1	12.8

*NS: Non Symmetric: Pin at one end and roller at the other end.

S: Symmetric: Both ends pinned.

decided to consider both the boundary conditions in the analysis of the bridge under the application of horizontal loads (Chapter 4).

BEHAVIOR OF COMPOSITE I-BEAM BRIDGES SUBJECTED TO HORIZONTAL LOADS

4.1 INTRODUCTION

Composite girder systems are often subjected to forces applied parallel to the plane of the slab. For this type of loading the girder slab system is subjected to torsion as well as normal forces and bending moments. Consequently, the cross section of the bridge is expected to warp and the assumption made in the plate theory that plane sections normal to the plate remain plane is invalid. Therefore, the use of the orthotropic plate theory to solve the problem of horizontal loads is highly questionable. Gustafson's model, where the nodes of the slab and the beam elements lie in one plane, is also based on the same basic assumption of no warping of the cross section and hence would be of limited use. One approach to the problem is to treat the composite girder system as a beam subjected to torsion and bending moment. An analysis similar to Chu and Longinow (48) could then be used to determine the shear stresses in the section due to torsion. The results of this solution can then be superimposed on a solution of the bending problem. This method, however, would neither include the influence of diaphragms and lateral bracing in the distribution of the loads, nor would the method yield information about the displacements and forces in the diaphragms, which are of primary interest.

It is believed that the finite element model developed in the second chapter provides an accurate method for determining the forces and

moments in the diaphragms and lateral bracing. Also, the analysis gives insight into the response of the slab and girders to the loads applied parallel to the plane of the slab.

4.2 SUMMARY OF THE BRIDGES CONSIDERED IN THE ANALYSIS

The bridge shown in Figure 7 is selected to study the effects of the horizontal loads on the composite I-beam bridge. The bridge is simply supported over a span of 100'. The analysis of the bridge under the influence of sine loading was carried out in Chapter 3 to compare the present approach with Gustafson's finite element model and Wright's thin beam approach. It is believed that the role of the diaphragms is very apparent in such a bridge because of its deep longitudinal girders.

The influence coefficients for the deflections, shears and moments, etc., for the bridge are obtained for the unit horizontal load acting from one end of the bridge to the other. The loads are applied at the node points of the top and the bottom flanges of the exterior girder A.

In order to analyze the role of the diaphragms and to study the changes in the behavior of the bridge due to the inclusion of the diaphragms, the bridge is solved without diaphragms in one case and with the diaphragms at the end sections of the bridge and at an interval of one-quarter span.

The study includes three kinds of diaphragms: beams, axial bars and plate diaphragms. The beam diaphragms at each section are represented as an assemblage of six beam elements, two elements between each pair of

longitudinal girders (Figure 11a). The beams used are 27WF84. The nodes of the beam elements are assumed to lie in the middle plane of the slab. The stiffness matrix for the beam element is obtained as explained in section 2.2.2, which accounts for the eccentricity of the beam nodes.

Bar diaphragms are assumed to have nodes which coincide with the nodes of the top and the bottom flanges of the longitudinal girders. Three bars, each of 4" x 4" x 1/2" angle section, are provided between each pair of longitudinal girders (Figure 11b). Two of them are inclined, running from the top flange of one girder to the bottom flange of the adjoining girder, and the third bar joins the bottom flanges of the two girders.

In the third case diaphragms are treated as plates. The dimensions of the plates are obtained so that the moment of inertia of the plate section is equal to the moment of inertia of the beam diaphragms. The plate diaphragms are assumed to be as deep as the longitudinal girders and their nodes coincide with the nodes of the top and the bottom flanges of the longitudinal girders (Figure 11c). Each node of the plate diaphragm has six degrees of freedom U , V , W , θ_x , θ_y , and θ_z , as is the case with the other plate elements of the bridge. The plate diaphragms are represented by three plate elements at each cross section, one between each pair of longitudinal girders.

The study also considers two different support conditions for the bridge. In one case the girders of the bridge are pinned at one end ($x=0$) and are supported on rollers at the other end ($x=L$), whereas, in the second

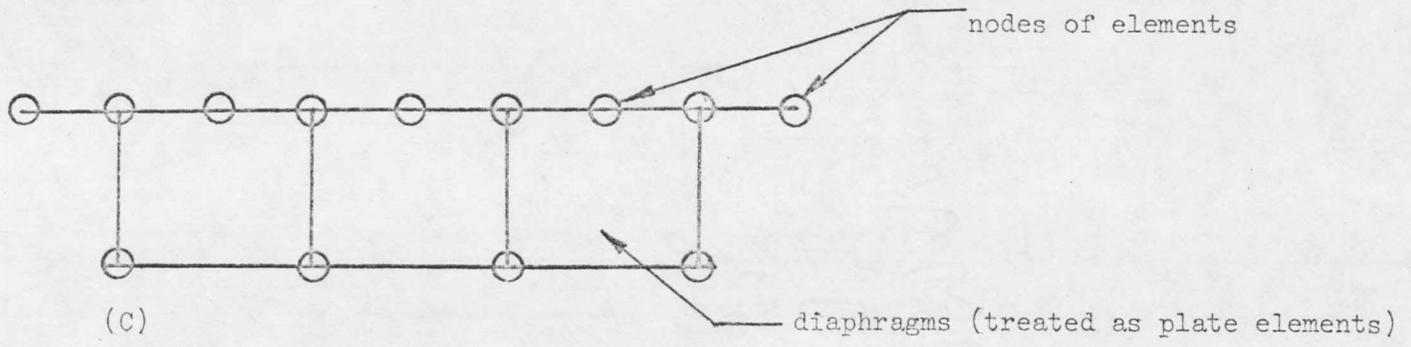
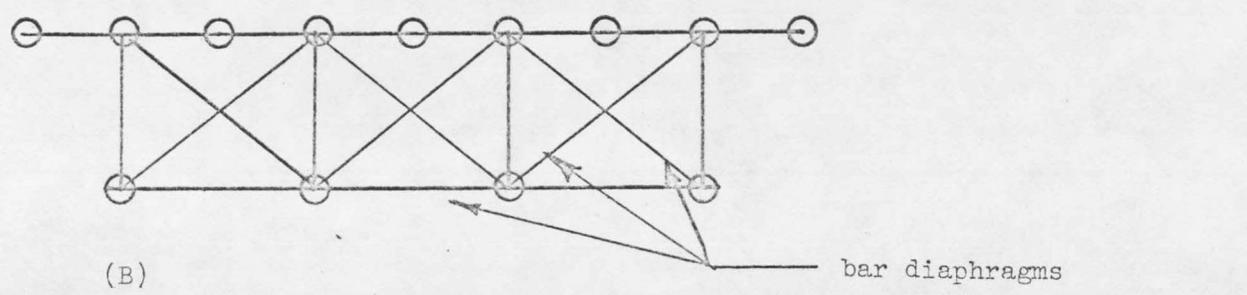
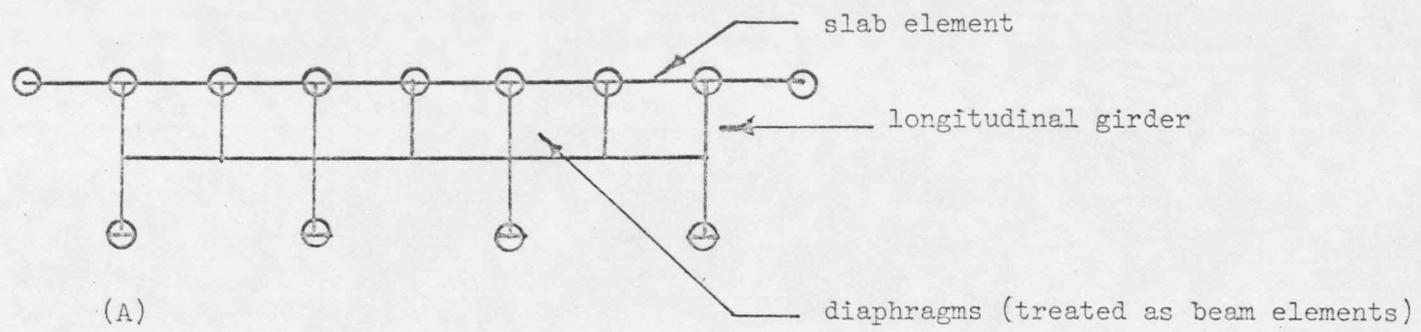


Figure 11. Various kinds of diaphragms.

case both ends of the girders are pinned. The study of the bridge for these boundary conditions is undertaken because of the substantial differences which were observed in the distribution of the vertical loads for the two cases (Chapter 3).

After studying the influence of the diaphragms for the above bridge, it was desired to determine if the effect of the diaphragms remains essentially the same for a bridge with different span and girder size. The bridge selected for this purpose is a four girder bridge pin supported over a span of 76'. Its longitudinal girders are 4' deep (Figure 12). The bridge is solved for the horizontal loads acting at the top and the bottom flanges of the exterior girder A. In one case the bridge is provided with bar diaphragms at quarter span intervals and at the end sections. In the other case it does not have any diaphragms.

In order to study the role of the intermediate diaphragms on the behavior of the bridge in Figure 7, it is also solved with the bar diaphragms provided only at the end sections of the bridge. For this case the bridge is symmetrically supported.

A summary of the various types of diaphragms, load positions and the boundary conditions of the bridges investigated is presented in Table VIII.

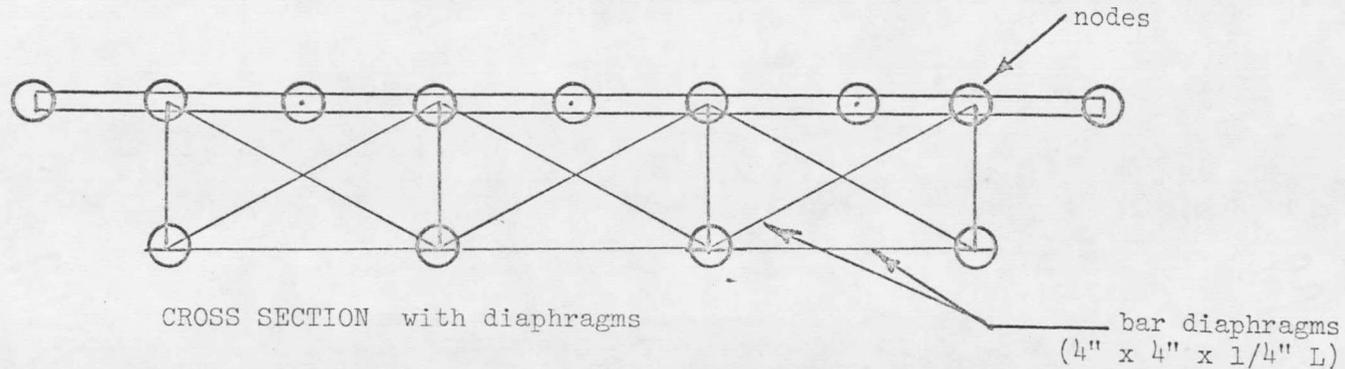
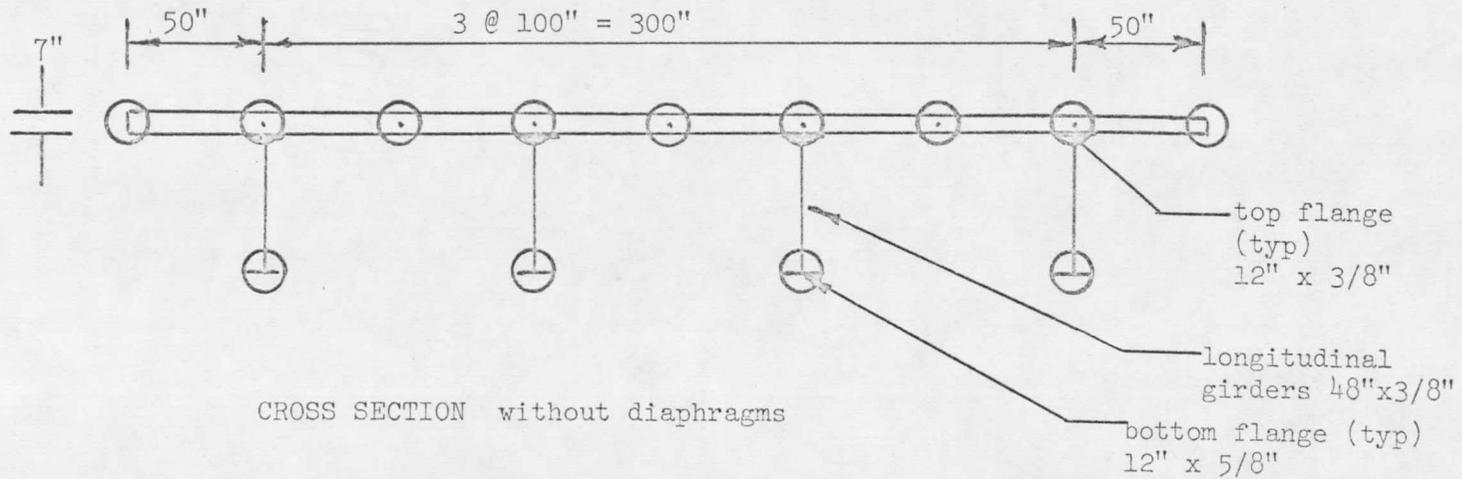


Figure 12. Details of 76' span four girder bridge.

TABLE VIII. Summary of the Bridges Studied

Bridge	Type of Diaphragm	Position of Load	Boundary Condition	Identification #
100' span, 4 girder bridge	no diaphragm	nodes of the	nonsymmetric	UNB6
	Beam	upper flange	pinned at X = 0	UNA6
	bar (axial)	of girder A	supported on	UNB6
	plate		rollers at X=L	UNA6
	bar (axial)	upper flange	symmetric	USA6
	no diaphragm		(both ends pinned)	USN6
	beam	lower flange	nonsymmetric	LNN6
bar (axial)			LNB6	
	plate		LNA6	
			LNP6	
	no diaphragm	lower flange	nonsymmetric	LSN6
	bar (axial)			LSA6
	bar (axial)	upper flange	symmetric	USA6*
	bar (axial)	lower flange	symmetric	LSA6*
76' span, 4 girder bridge	no diaphragm	upper flange	symmetric	USN4
	bar (axial)			USA4
	no diaphragm	lower flange	symmetric	LSN4
	bar (axial)			LSA4

- # First letter indicates load position U:upper, L:lower.
 Second letter indicates type of supports S:symmetric, N:nonsymmetric
 Third letter indicates type of diaphragm N:no diaphragm, B:beam, A:bar (axial), P:plate
 Last number indicates depth of girders in feet
 * indicates diaphragms are provided only at the end sections of the bridge.

4.3 EFFECT OF DIAPHRAGMS ON BRIDGE DEFLECTIONS

For the analysis of the bridges the coordinate system used is shown in Figure 6. Loads are applied in the negative y direction acting toward the interior girders. Influence coefficients are obtained for the deflections, shears and moments.

The two boundary conditions considered are:

- (1) Nonsymmetric boundary conditions: Here the bottom flanges of the longitudinal girders are supported on pins at $x = 0$ which restrain longitudinal, transverse and the vertical motion of the supported end. The flanges are supported on roller bearings at $x = L$ which permit the longitudinal extension of the bottom flange.
- (2) Symmetric boundary conditions: Here both ends of the longitudinal girders are pinned.

Tables A1 to A12, in Appendix 2, show influence line ordinates for the longitudinal, transverse and vertical deflections of the longitudinal girders. The coefficients are obtained for the different types of diaphragms and for the symmetric and nonsymmetric boundary conditions.

From these tables we observe: In the absence of the diaphragms or when the diaphragms are treated as beam elements whose nodes coincide with the nodes of the slab elements, the deflections of the bridge in the longitudinal and vertical directions are negligible in comparison to the transverse deflections. The longitudinal and the vertical deflections of the various nodes of the slab are about 0.1 to 2.0 percent of the transverse

deflections of the slab. The transverse deflections are drastically reduced when the bridge is provided with bar or equivalent plate diaphragms. The influence line ordinates for longitudinal and vertical displacements are also reduced. For this bridge all three displacement components are of the same order of magnitude.

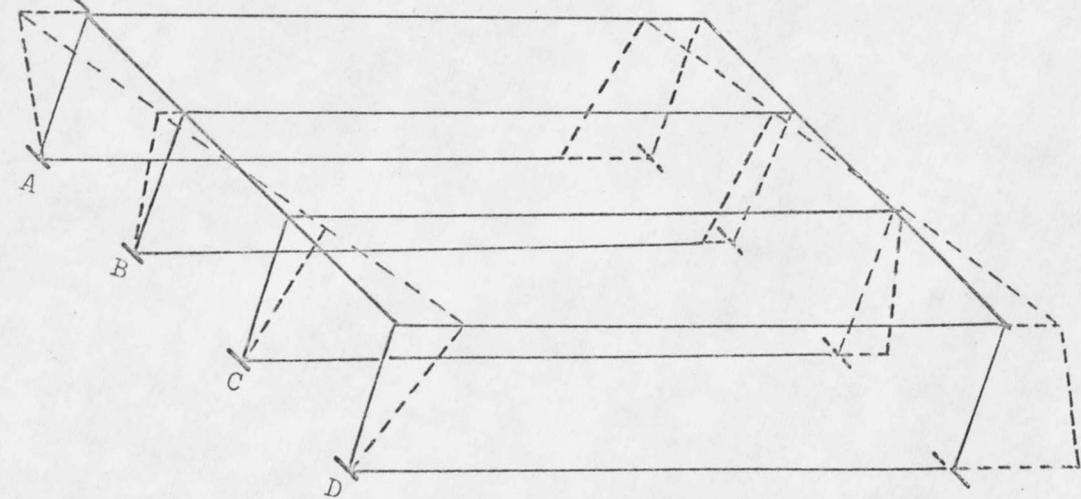
4.3.1 Longitudinal Deflections

Figure 13 is a sketch showing exaggerated longitudinal, transverse and vertical deflections of the bridge without diaphragms. For this case the unit horizontal load at $x = 0$ was applied on the top flange of the exterior girder A. From the figure we note that the end sections of the slab rotate about the vertical axis. The behavior is easily explained by replacing the horizontal load with a couple about the vertical axis and a load at the midspan section of the top flange of girder A. The load now acting at the midspan of girder A causes essentially a rigid body displacement of the slab in the transverse direction since the web plates of the longitudinal girders are flexible in the transverse direction. The couple tends to cause a rotation of the bridge about the vertical axis. Since the girders are supported on both the ends at the bottom flange, and since the longitudinal girders distort easily in the absence of the diaphragms, the net result is the rotation of the slab in the direction of the couple. When the load is applied between $x = 0$ and $x = L/2$, the couple about the vertical axis is positive (clockwise). Its magnitude decreases as the load approaches midspan of girder A. When the load is on the other side of the midspan, the direction of the couple is reversed and hence the sign of the

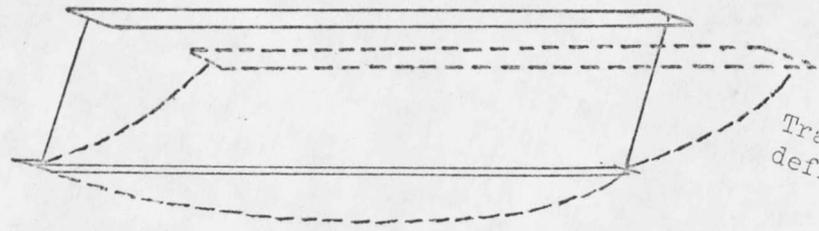
Longitudinal deflections

82

load position



Transverse deflections



Vertical deflections

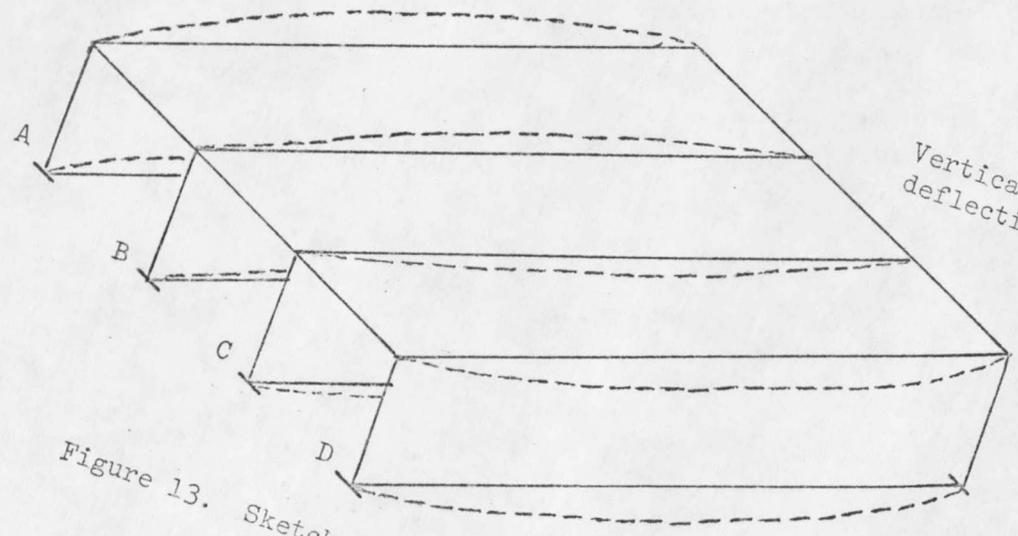


Figure 13. Sketch of deflections of no diaphragm bridge for unit load on the top flange of girder A.

longitudinal deflections are changed.

When bar diaphragms or equivalent plate diaphragms are provided in the bridge the deflected shape of the bridge changes significantly. Now the load is distributed to the supports through the end diaphragms and not by the distortions of the longitudinal girders. The end diaphragms serve as the bracing and prevent excessive transverse deflections of the slab.

When the load is applied at $x=0$ on the top flange of girder A, the slab of the bar diaphragm bridge rotates about the vertical axis. The behavior is similar to the behavior of the no diaphragm bridge. The longitudinal displacements for the bar diaphragm bridge are smaller than the displacements of the no diaphragm bridge. When the load is applied at the midspan of the bridge, the behavior of the slab is similar to a beam supported on springs, i.e., the top flange of the loaded girder A is in compression and the unloaded exterior girder D is in tension, and the transverse deflections of the slab are parabolic* with maximum deflections near the center of the slab.

When the loads are applied at the bottom flange of girder A, the longitudinal displacements of the bridge deck are smaller for the no diaphragm bridge than for the bar diaphragm bridge. The displacement patterns are similar to the deflected shape of the bar diaphragm bridge subjected to loads at the top flange of girder A. In other words, the top flange of the loaded girder is in compression and the top flange of the

* The term parabolic in this chapter is used to indicate a curve with single curvature.

unloaded exterior girder is in tension. The smaller longitudinal deflections of the no diaphragm bridge are attributed to the inefficient transfer of the load from the loaded girder to the unloaded girders. The loaded girder A undergoes considerable transverse deformations in order to transfer the load to the deck and hence to the unloaded girders. As may be expected the amount of compression and tension in the top flanges of the exterior girders increases as the load moves closer to the midspan and decreases when loads move away from the center of the span. However, the magnitude of the longitudinal deflection of the end sections does not necessarily follow this trend. This trend is seen to hold in the non-symmetrically supported bridge with the bar or the plate diaphragms. It, in general, does not hold for the no diaphragm bridge regardless of the boundary conditions.

4.3.2 Transverse Deflections

As mentioned before, the transverse displacements for the slab and the top flanges of all the girders of the no diaphragm bridge or the beam diaphragm bridge are essentially constant along the span of the bridge when the horizontal loads are applied at the upper flange of girder A. A small decrease in the transverse displacements which is observed between the loaded end of the girder and the unloaded end of the girder is attributed to the small transverse stiffness of the deep and thin webs of the longitudinal girders. The nodes of the top flange deflect in a straight line. A careful examination of the deflections shows that they satisfy Maxwell's reciprocal relationship. The motion of the slab is essentially

a rigid body translation. The behavior is easily ascribed to the lack of the cross bracing which connects the slab to the bottom flanges of the girders and the supports of the bridge. It is observed that most all the rigid body translation of the slab disappears when bar diaphragms or equivalent plate diaphragms are included in the bridge. For this case, when the load is applied at $x=0$ or $x=L$, the transverse deflections of the nodes of the top flanges vary as a straight line along the longitudinal axis. When the load is applied at the midsection of girder A, the transverse deflections are parabolic. The deflected shape is similar to the deflected shape of the transversely loaded beam.

The bottom flanges of the girders deflect parabolically in the transverse direction when the no diaphragm bridge is acted upon by a unit horizontal load at the top flange of girder A. The deflections of the bottom flanges of all the girders are equal with the maximum ordinate being about three fourths of the rigid body displacement of the slab. The behavior is expected since both ends of the girders are pinned against motion in the transverse direction.

When the loads are applied at the bottom flange of girder A instead of at the top flange, the behavior of the no diaphragm bridge is slightly changed. Here the bottom flanges of the girders deflect parabolically in the transverse direction. The magnitude of transverse deflections of the loaded flange is about five times that of the unloaded flanges. The slab deflects essentially as a rigid body. The amplitude of motion is approximately 40% smaller than when the load is applied on the top flange.

When the loads are applied at the bottom flange of girder A of the bridge with the bar or the plate diaphragms, the deflected shape of the slab remains the same as when the loads were applied at the top flange of the girder. The transverse deflections of the loaded bottom flange are larger than the transverse deflections of the bottom flanges of the unloaded girders. The ratios of the deflections of the loaded bottom flange to the unloaded bottom flange, as may be expected, are not as large as in the case of the no diaphragm bridge.

When the boundary conditions of the bridge are changed from nonsymmetric to symmetric, the deflected shapes of the bridges remain unaltered except for the slightly smaller amplitudes of the transverse deflections of the slab of the no diaphragm bridge.

4.3.3 Vertical Deflections

As was mentioned before, the vertical deflections of the bridge with or without the diaphragms are of the same order of magnitude as the longitudinal deflections. Girders A and B of the no diaphragm bridge or the beam diaphragm bridge deflect upwards when the unit horizontal load is applied on the top flange of girder A (and the load is between $x=0$ and $x=L/2$) whereas, girders C and D deflect downwards.

When the load was acting on the bottom flange of girder A the exterior girders A and D deflect downwards and the interior girders B and C deflect upwards. Such deflection configurations indicate that the bridge behaves somewhat like a plane frame lying entirely in the yz plane. The added.

stiffness of the slab in the presence of the beam diaphragm may be noted from the slightly smaller vertical deflections of the beam diaphragm bridge than the no diaphragm bridge. The differences in the deflections are greater when the loads are applied on the bottom flange than when they are acting on the top flange.

In contrast to the behavior of the no diaphragm bridge, girders A and B deflect up when bar or plate diaphragms are provided in the bridge (except when the load is acting at $x=0$). This behavior may be explained, if the bridge is assumed to be a simply supported beam with a cross section consisting of a channel with four flanges. The shear center for such a beam would lie above the slab. Hence, when a unit horizontal load is applied at the top or the bottom flanges of girder A it causes a torsional moment about the longitudinal axis. This moment would tend to lift girders C and D. The magnitudes of the deflections are observed to be greater when the unit load is applied at the bottom flange of girder A rather than the top flange. This is because of the greater torsional moments in the former case.

The deflected shapes for both the no diaphragm and the bar diaphragm bridges are parabolic with the maximum ordinates occurring in the vicinity of the midsection. Also, the top and bottom flanges of the girders deflect almost equally, which implies that there is no stretching or shortening of the web plates of the girders.

When the boundary conditions are changed from nonsymmetric to

symmetric for the no diaphragm bridge subjected to transverse loads, there is a marked reduction in the magnitudes of the longitudinal and vertical deflections. The longitudinal motions are reduced almost by a factor of ten and the vertical deflections are reduced by a factor of four. The reductions in the magnitudes vary with the load position and the point on the span at which the comparison is made. The reductions in the transverse deflections are almost negligible.

The reductions in longitudinal, transverse and vertical deflections of the bar diaphragm bridge, when the boundary conditions are changed from nonsymmetric to symmetric are smaller for the top and bottom flange loadings than the reductions noted in the no diaphragm bridge.

Besides the reductions in the magnitudes of the longitudinal deflections of the no diaphragm and the bar diaphragm bridge, there is a considerable change in the longitudinal deflected shape of the bottom flange. When the load is applied at the midspan section of girder A the distribution of the longitudinal deflections resemble a complete sine wave with zero longitudinal deflection at the midsection and the end supports. Alternately, the bottom flange on either side of the midsection moves towards or moves away from the midsection.

In conclusion, it is noted that the no diaphragm bridge undergoes considerable warping when subjected to horizontal loads. Beam diaphragms whose nodes are treated as in the present investigation are essentially of no help in preventing the warping of the cross section. When bar diaphragms

or equivalent plate diaphragms are provided, the response of the bridge to the horizontal loads is significantly changed and the magnitudes of the deflections are considerably reduced.

4.4 EFFECT OF DIAPHRAGMS ON GIRDER REACTIONS

As was mentioned in the previous section the four girder composite I-beam bridge, supported over a span of 100', is solved for the various combinations of the load positions, boundary conditions and type of diaphragms. The longitudinal, transverse and the vertical reactions for the bridge are tabulated in Tables A13 to A15 (Appendix 2). These tables provide a comparison between the load distribution characteristics of the no diaphragm bridge with the diaphragm bridges. It may, however, be noted that the reactions of the bridge are interwoven with the deflected shape of the bridge and hence the discussion of the reactions is impossible without reference to the deflections (Tables A1 to A12).

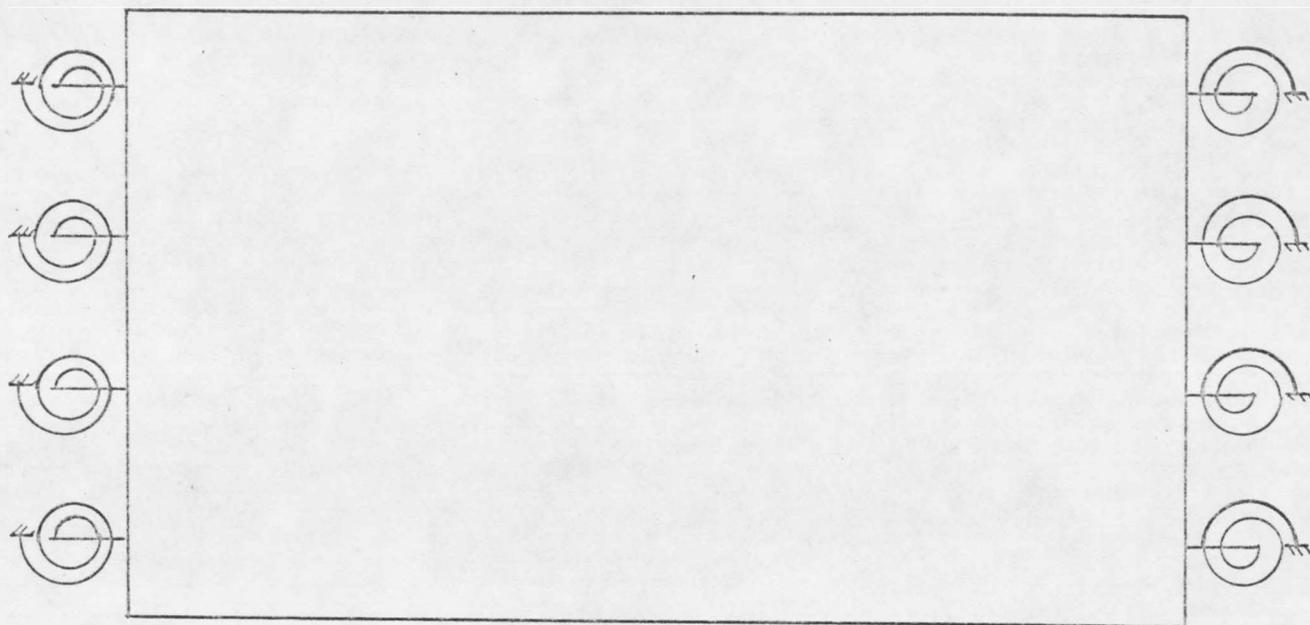
From Tables A13 to A15, one sees a striking influence of the bar and the plate diaphragms on the bridge reactions. When the unit horizontal load is applied on the upper flange of the exterior girder A, the longitudinal reactions for both the symmetric and nonsymmetric boundary conditions are much smaller for the bridge with the bar diaphragms or the plate diaphragms than the reactions of the bridge without diaphragms or with the beam diaphragms. The longitudinal reactions for the plate diaphragm bridge are somewhat greater than the corresponding reactions of the bar diaphragm bridge. The reactions for the beam diaphragm bridge and the no diaphragm bridge are equal in magnitude. When the load is applied between

$x=0$ and $x=L/2$, the reactions of the exterior girders A and D are slightly larger for the nonsymmetrically supported beam diaphragm bridge than those for the no diaphragm bridge. Whereas, they are a little smaller than the corresponding reactions of the no diaphragm bridge when the load is applied between $x=L/2$ and $x=L$. An opposite trend is observed when the reactions of the interior girders are compared.

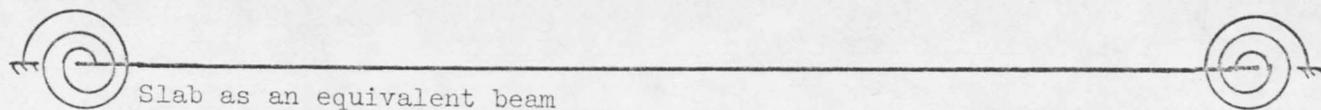
In the case of the no diaphragm bridge or the bridge with beam diaphragms, the longitudinal reactions of the girders are considerably larger than the transverse and vertical reactions of the girders when the loads are applied on the top flange of the exterior girder A. The longitudinal reactions, though small when the bar diaphragms or the plate diaphragms are provided in the bridge, are of comparable magnitudes to the transverse and the vertical reactions.

4.4.1 Longitudinal Reactions

The existence of the longitudinal reactions due to the application of transverse loads is rather a complex phenomenon. It may, however, be explained qualitatively by the simplified beam of Figure 14. Here, the slab of the no diaphragm bridge is treated as a beam. The "beam" is supported on its two ends by rotational springs which represent the stiffness of the longitudinal girders along the transverse direction. The restraining moments of the slab, if any, are provided by the longitudinal reactions of the girders. The slab also transfers the longitudinal shearing forces to the girders in order to maintain the static equilibrium of the moments about the z axis.



Bridge as a beam supported on rotational springs



Slab as an equivalent beam

Figure 14. Equivalent 'beam slab'.

As mentioned in section 4.3, the slab of the no diaphragm bridge translates essentially as a rigid body when the transverse load P is applied on the top flange of girder A. This in turn results in equal transverse reactions for all the supports of magnitude $0.125P$ (approximate). These reactions, when the load is not applied at the midspan of girder A, cause an unbalance of moments about the vertical axis and hence require longitudinal reactions for the static equilibrium of the moments. When the load is applied at the end sections ($x=0$ or $x=L$), the longitudinal reactions are solely due to the transfer of the shear. Whereas when the load is acting at the midspan section of the bridge, the longitudinal reactions are not required for static equilibrium of moments about the z axis. However, the longitudinal reactions in this case are required to provide the restraining moments of the 'beam slab'. The restraining moments and therefore, the longitudinal reactions for the 'beam slab' would be considerably larger if the rotational springs were not provided to represent the transverse stiffness of the girders. When the load is acting anywhere between the midspan and the end sections, the longitudinal reactions are caused by both the shear transfer (for static equilibrium of moments) and the restraining moments of the beam slab. When both ends of the girders are pinned, the total longitudinal reactions for the exterior girders remain the same as for the case of nonsymmetric boundary conditions. However, the total longitudinal reaction of the girders is shared between its two ends at $x=0$ and $x=L$.

The exterior girders share a large percentage of the total longitudinal

reaction when the load is not applied at the midspan section of the top flange of girder A. But, when the load is applied at the midspan of girder A the interior girders of the no diaphragm bridge resist a larger portion of the longitudinal reaction than the exterior girders of the bridge.

As mentioned previously, the slab of the bridge with bar diaphragms, when loaded at the top flange of girder A, does not translate as a rigid body but deflects more or less like a simply supported beam. Hence the transverse reactions in this case are not a fixed value for different longitudinal positions of the load, as was the case with the no diaphragm bridge girders. The influence lines of the transverse reactions are observed to be straight lines (see Figure 15). The longitudinal reactions which result in this case are difficult to explain with the previous model where the slab is treated as a beam supported on the rotational springs. It may be seen from Figure 16 that the influence lines for the longitudinal reactions of the bar diaphragm bridge are not straight lines. Whereas, they are straight lines for the no diaphragm bridge (Figure 17).

When the loads are applied on the bottom flange of the longitudinal girder A, the distribution of the longitudinal reactions is completely changed. The magnitudes of the longitudinal reactions are substantially reduced for the loaded girder of the no diaphragm bridge. The magnitudes of reactions are even smaller for the loaded girder of the beam diaphragm bridge. The behavior is partly explained by the fact that for this case the loaded girder undergoes considerably more transverse deflections than

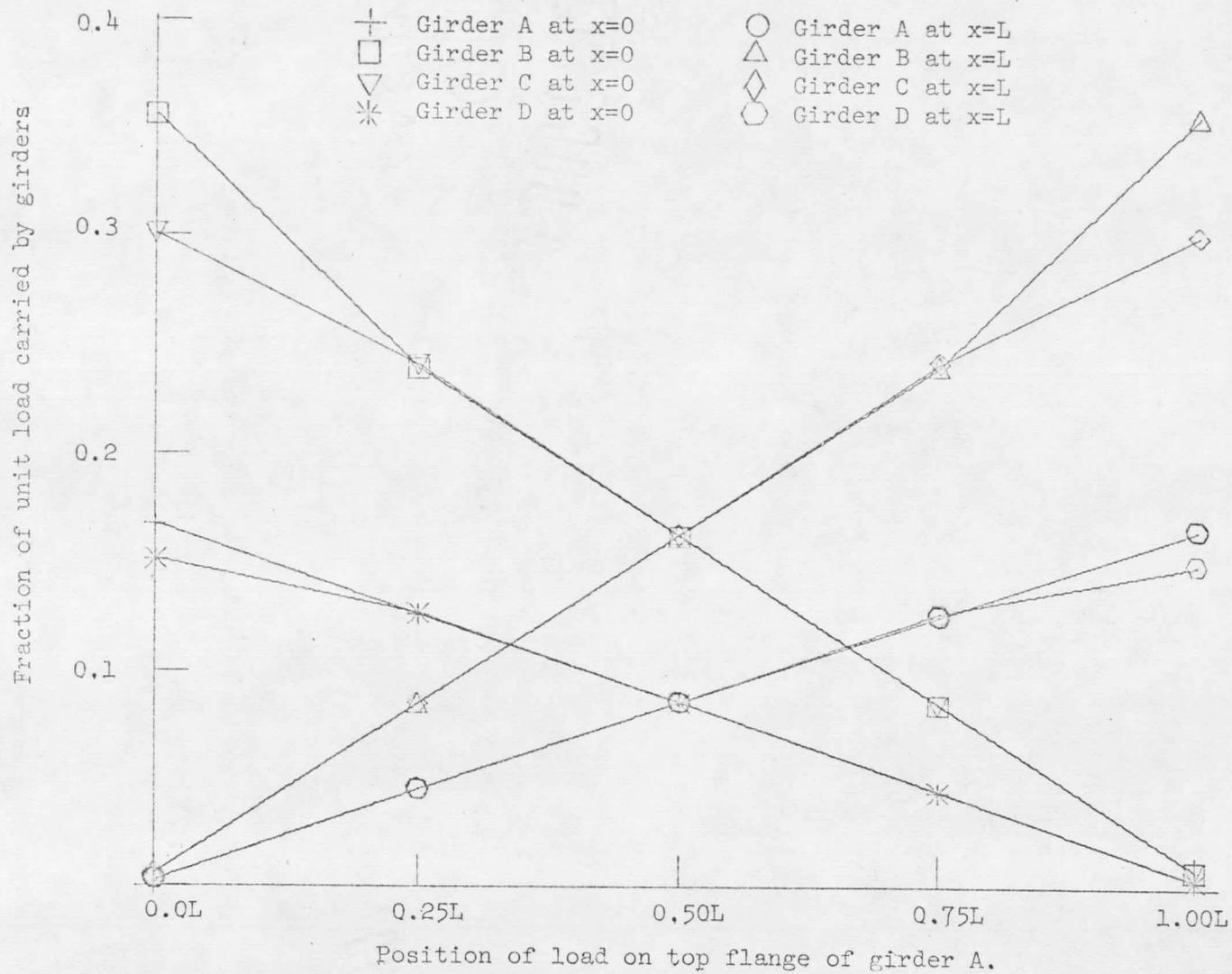


Figure 15. Influence lines for transverse reactions (bar diaphragm bridge, non-symmetrically supported).

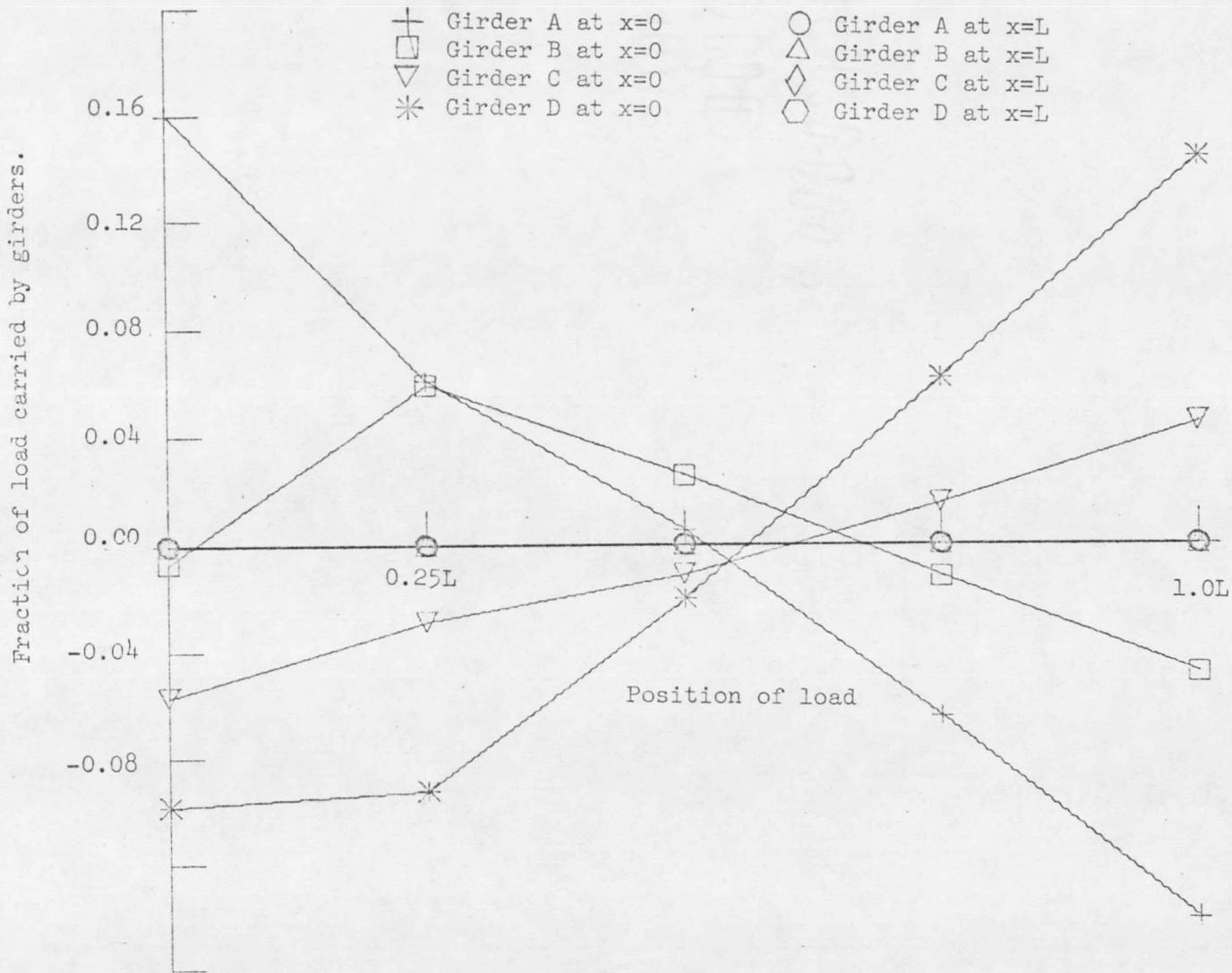


Figure 16. Influence lines for longitudinal reactions (bar diaphragm bridge, non-symmetrically supported), (load applied on top flange of girder A).

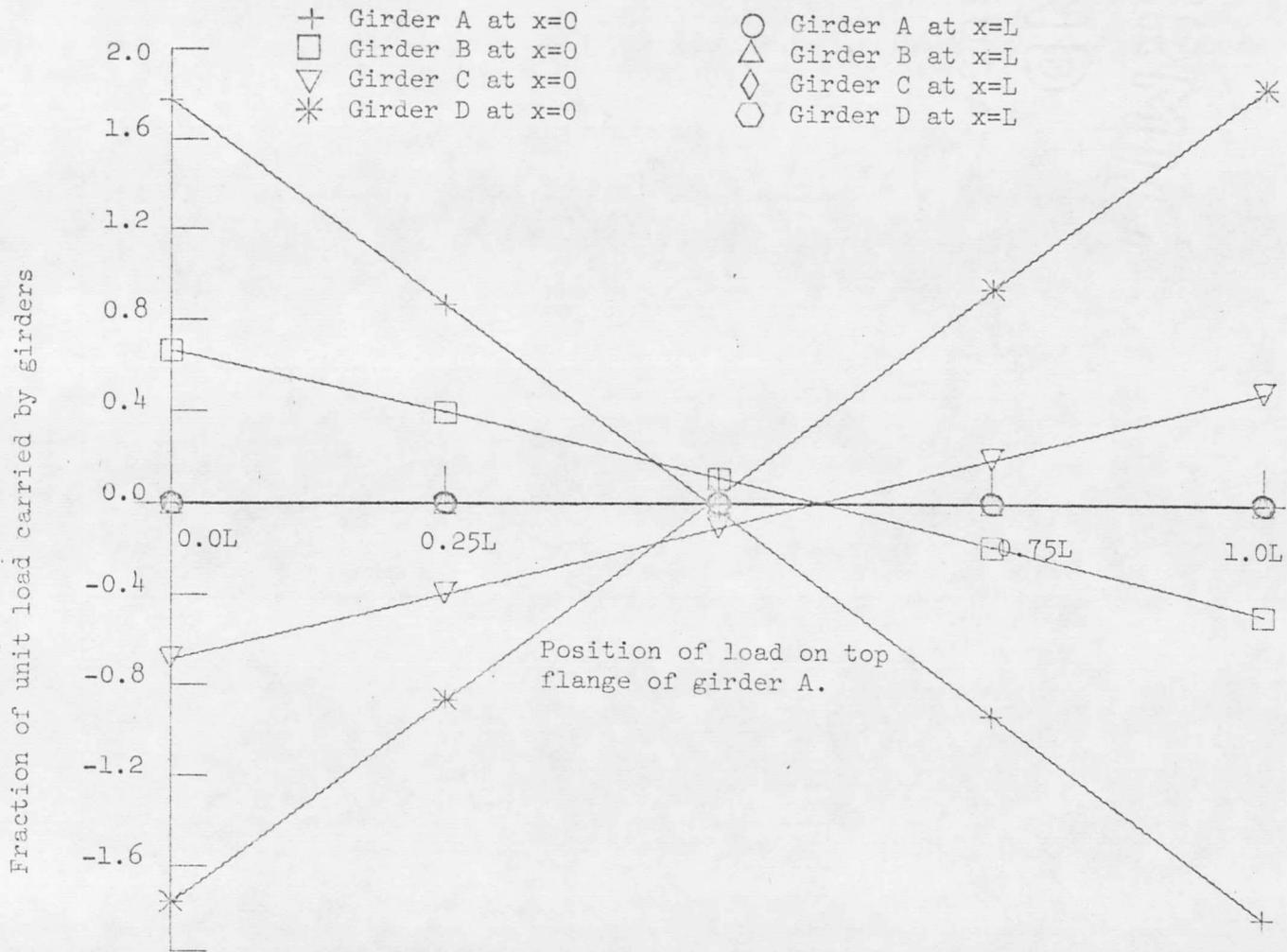


Figure 17. Influence lines for longitudinal reactions (no diaphragm bridge, non-symmetrically supported).

the unloaded girders, and hence, resists a large portion of the applied load. The remaining portion of the applied load is resisted equally by the remaining six supports of the three unloaded girders. These transverse reactions do not form as much an unbalance of moments about the vertical axis as was the case with the top flange loading. In contradiction to the above behavior of the no diaphragm bridge or the beam diaphragm bridge, the longitudinal reactions are increased for the case of the bridge with the bar or the plate diaphragms, when the loads are applied on the bottom flange instead of the top flange.

4.4.2 Transverse Reactions

Focusing our attention to the transverse reactions of the girders for various types of diaphragms, we observe that when the load P is applied on the top flange of girder A, the transverse reactions for all girders of the simply supported, no diaphragm, or the beam diaphragm bridge, are almost constant, $0.125P$, for various position of the load on the top flange of girder A. The behavior is in agreement with the deflected shape of the bridge. The small variations in the reactions which are noted between the two ends at $x=0$ and $x=L$ are ascribed to the small stiffness of the longitudinal web plates in the transverse directions. When the bar diaphragms or the plate diaphragms are provided at an interval of the quarter-span of the bridge, the stiffness of the girders in the transverse direction increases and the slab does not deflect like a rigid body but behaves more or less like a simply supported beam. This may be verified by adding the reactions for all four girders at $x=0$ and comparing the

total reaction with the reactions of the simply supported beam.

The influence line diagrams for the transverse reactions, as may be expected for a simply supported beam, are straight lines (Figure 15). They also are symmetric about the longitudinal axis passing through the center of the bridge. It may also be observed from Figure 15 that the magnitude of the reactions of the interior girders are almost twice as much as the transverse reactions of the exterior girders.

There is no appreciable change in the transverse reactions when the boundary conditions are changed from nonsymmetric to symmetric. For the case of the no diaphragm bridge, the distribution of the reactions is a little more uniform, i.e., the differences in the transverse reactions between the two ends of the girders are smaller for the symmetric boundary conditions than the differences in the reactions for the nonsymmetric boundary conditions. In the case of a bridge with the bar diaphragms, a small reduction in the transverse reactions is noted at $x=0$ when the bridge is symmetrically supported. This decrease gets smaller as the load moves from $x=0$ to the midspan of the top flange of girder A and it changes sign when the load is at midspan.

When the loads are applied on the bottom flange of the exterior girder A, the distribution of the transverse loads to various girders is altered considerably. For the bridge with no diaphragms or with the beam diaphragms the loaded girder has a larger share of the applied load. The remaining three unloaded girders share the rest of the applied load

equally among them. The behavior seems logical when the deflected shape of the entire bridge is considered. The flange of the loaded girder A deflects parabolically in the direction of the load and transfers the load to the web plate. The slab, due to the small stiffness of the longitudinal girders, translates essentially as a rigid body in the transverse direction. This in turn, causes all three unloaded girders to deflect by an equal amount. And hence, these three girders share the remaining portion of the applied load in equal proportions. The contribution of the unloaded girders in resisting the applied load increases when the load gets closer to the midspan since the ratios of the transverse deflections of the unloaded girders to the deflections of the loaded girder become larger.

When the bottom flange of girder A of the bridge with the bar diaphragms or the plate diaphragms is loaded, the distribution of the reactions is slightly altered. The total reaction carried by each girder remains almost the same, but the distribution of the reaction between the two ends of each girder is changed. The reactions at $x=0$ have reduced for all the load positions and they have increased at $x=L$. Here again the interior girders carry twice as much reaction as the exterior girders and the reactions are symmetric about the longitudinal axis of symmetry.

When the boundary conditions are changed from the nonsymmetric to the symmetric, there is absolutely no change noted in the distribution of the transverse reactions of the no diaphragm bridge. A very slight redistribution of the reactions is noted for the bar diaphragm bridge. Here again, the total load carried by each girder remains almost the same but the

ends at $x=0$ carry greater reactions than the nonsymmetric case.

4.4.3 Vertical Reactions.

When the transverse load is applied on the top flange of girder A, it, along with the transverse reactions, creates a couple about the longitudinal axis which needs to be balanced by the vertical reactions.

The vertical reactions of the girders of the no diaphragm bridge and the beam diaphragm bridge are almost equal in magnitude. Similarly, the vertical reactions of the bar diaphragm bridge and the plate diaphragm bridge also have equal magnitudes. However, these magnitudes are completely different than those of the no diaphragm bridge. In the case of the bar diaphragm bridge, the vertical reactions of the support closer to the load resist the major portion of the total torque. On the contrary, the support closer to the load, for the no diaphragm bridge, resists a small portion of the applied torque.

The influence line diagrams for the vertical reactions of all four girders of the bridge with no diaphragms are straight lines (Figure 18). The behavior, though not plotted, is similar for the case of the bridge with the beam diaphragms. The influence line diagrams of Figure 18 show a fair degree of symmetry about the longitudinal axis of the bridge passing through the center of the slab. Similarly, one may observe, from Figure 19, that the influence lines for the vertical reactions of the exterior girders of the bridge with the bar diaphragms are straight lines. The influence lines for the reactions of the interior girders do not reveal any set pattern.

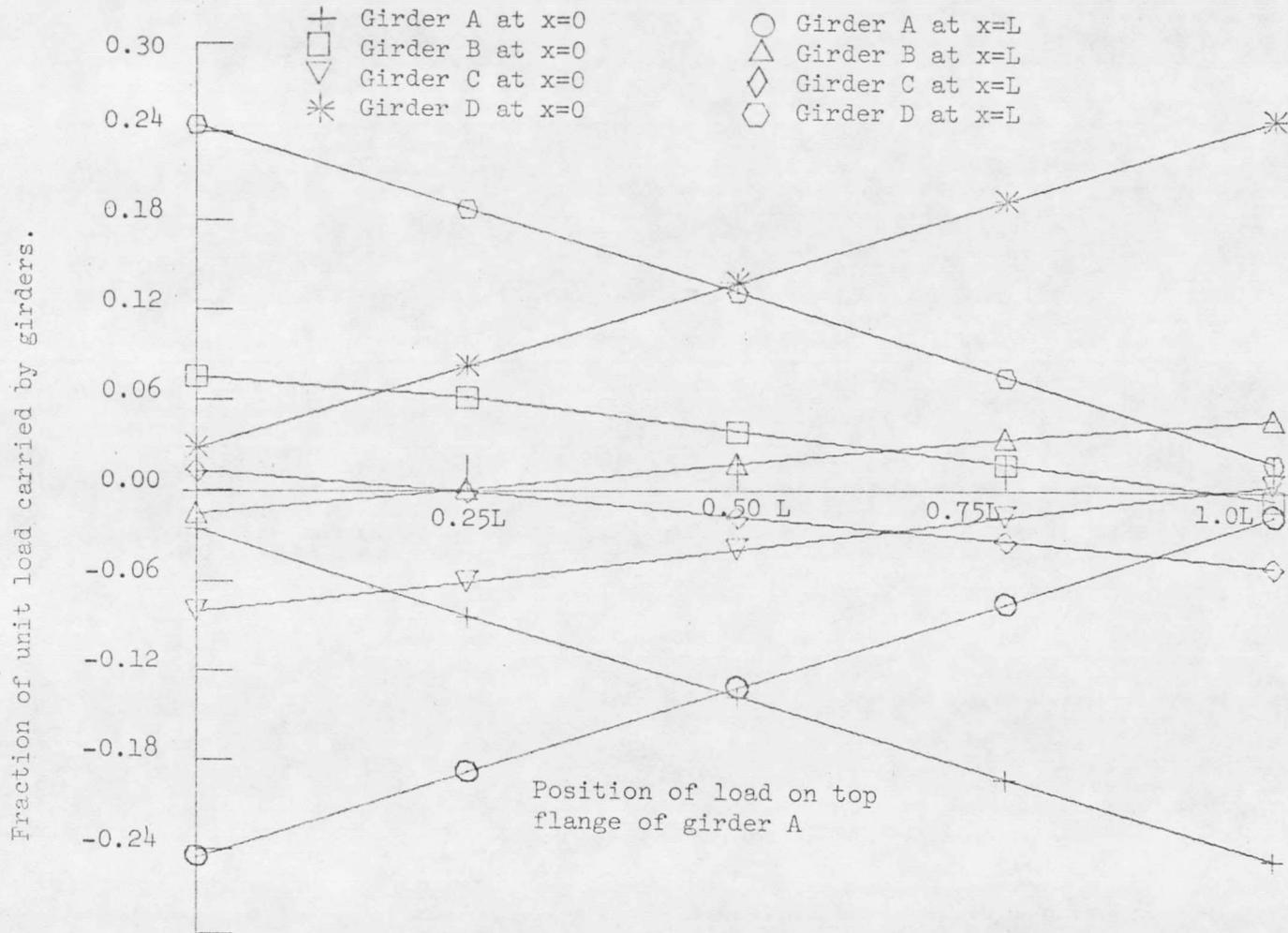


Figure 18. Influence lines for vertical reactions (no diaphragm bridge, non-symmetrically supported).

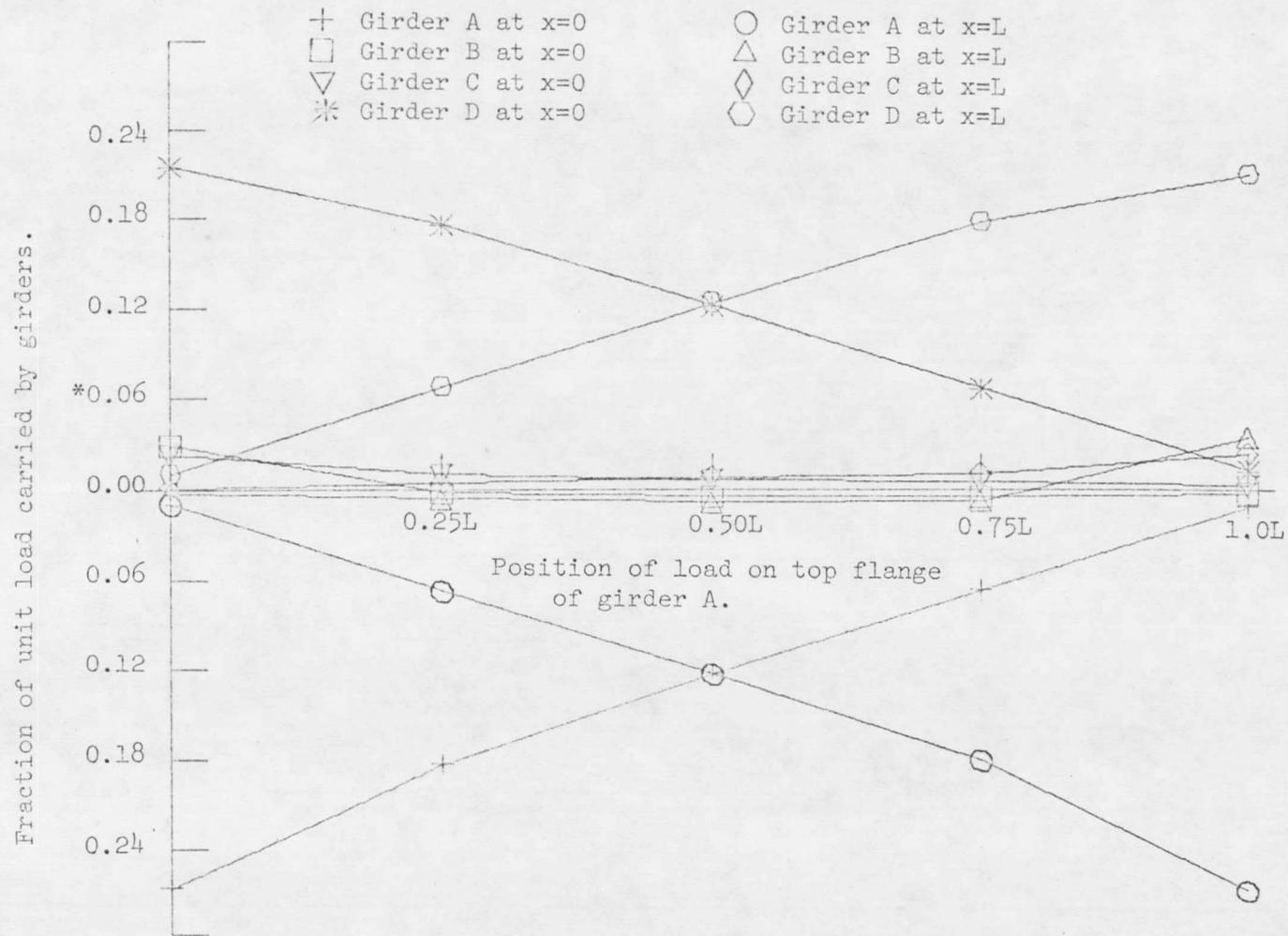


Figure 19. Influence lines for vertical reactions (bar diaphragm bridge, non-symmetrically supported).

When the loads are applied at the bottom flange of the exterior girder A, the vertical reactions are not needed for the static equilibrium of the forces and moments. Hence, the existence of the reactions is obviously due to the deformations of the longitudinal girders. Therefore, in order to appreciate the discrepancies which arise in the vertical reactions of the girders due to various types of diaphragms, one must resort to the deformations of the girders and not to the equilibrium conditions of statics.

From Table A15, one notes considerable changes in the reactions when the load is acting at the bottom flange instead of at the top flange. The direction of the vertical reactions of the loaded girders of the various bridges is changed at the $x=0$ end. The magnitudes of the reactions are also considerably different. The interior girders of the no diaphragm bridge have much larger reactions than when the loads were applied on the top flange of girder A. The distribution of the reactions between the two ends of the girders also has changed significantly. The differences in the reactions of the no diaphragm bridge girders and the beam diaphragm bridge girders are now increased. Similarly, the differences in the reactions of the plate diaphragm and the bar diaphragm bridge are also increased.

When the boundary conditions are changed from nonsymmetric to symmetric, we note a very small change in the vertical reactions for loads applied on the top flange of girder A. However, the changes are sizeable for the bar diaphragm bridge reactions when the bottom flange of girder A is loaded.

4.5 COMPOSITE BENDING MOMENT, M_y , FOR THE GIRDERS

In section 4.3 it was noted that some longitudinal and vertical curvatures are introduced when the transverse loads are applied on the top and the bottom flanges of girder A. These vertical curvatures produce moments, M_y , in the bridge. The magnitudes of these composite girder moments vary with the position of the load, the presence of the diaphragms in the bridge, the boundary conditions of the bridge and the girder under consideration. These moments, though not critical in most cases, may have considerable influence on the total composite girder moments when the bridge is subjected to vertical and horizontal loads, and hence may dictate significant alterations in the design of the composite girders.

Table A16 presents the coefficients of the composite girder moments, M_y , for all four girders of the bridge. The transverse loads are applied on the top and the bottom flange of girder A at quarter-span intervals. Table A16 also includes the coefficients of the composite girder moments for the simply supported no diaphragm bridge when subjected to vertical loads. Partial results are presented in Figures 20a to 20c, in the form of influence lines of the moment coefficients for girders A and B.

Several observations for the no diaphragm bridge can be made by comparing composite girder moments resulting from vertical and horizontal loads.

When the transverse load is applied at the quarter span section of girder A, the composite moment for girder A at the quarter span is

+ Girder A, UNN6
 ○ Girder B, UNN6

□ Girder A, UNA6
 △ Girder B, UNA6

◇ Girder A, Vertical load on
 * Girder B, -DO- girder A

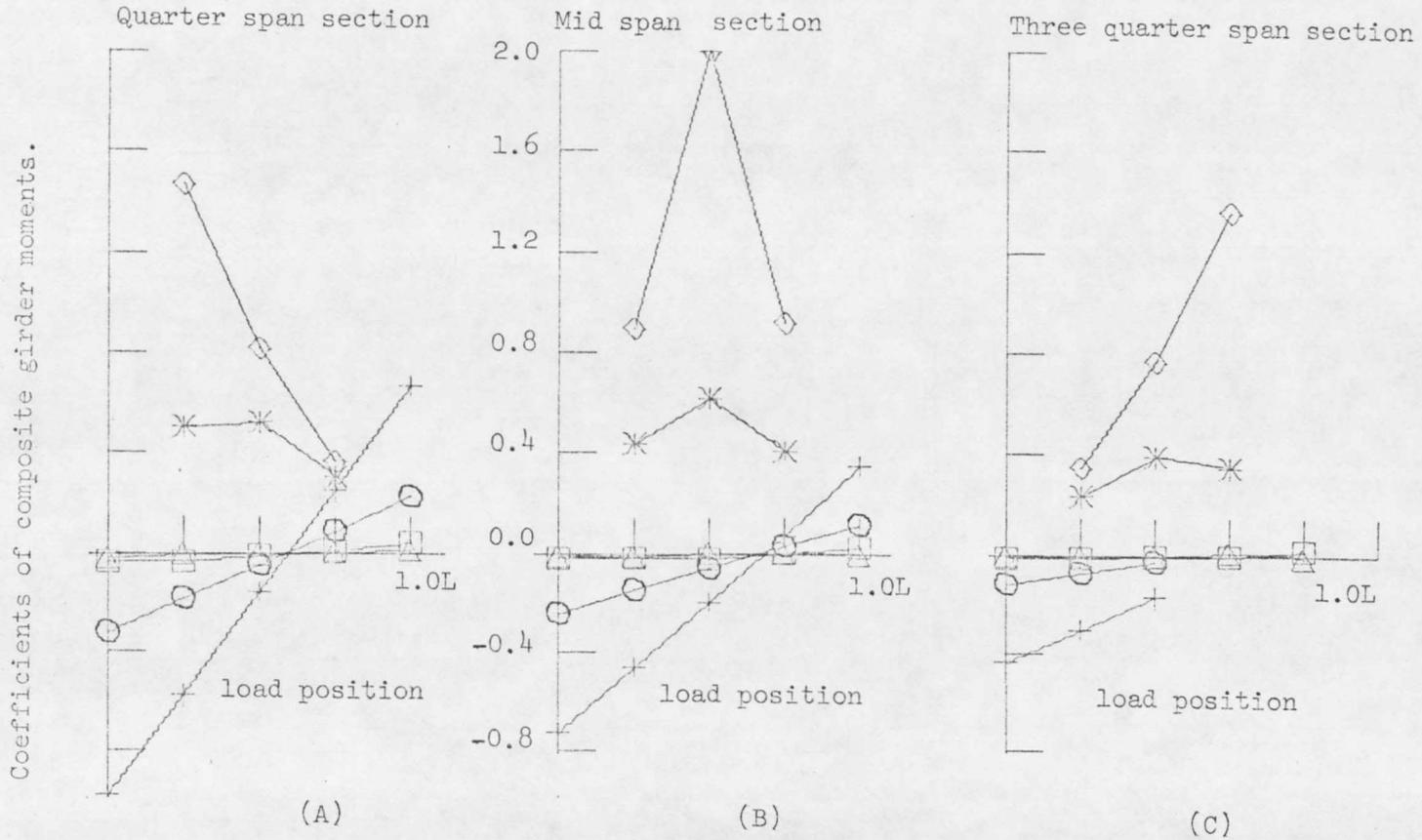


Figure 20. Influence lines for composite girder moments.

approximately one-third of the corresponding moment when the applied load is vertical. If the vertical load is applied on the quarter span section of girder B, the ratio of the moments at this section of the girder due to the transverse and the vertical loads is about one-fifth.

The moment coefficients for all four girders are greatest at the loaded section for both vertical and transverse loads. Their magnitudes decrease for sections away from the load. The reduction is more rapid for the vertical than for the horizontal loads.

When the vertical and the horizontal loads are applied at the midspan section of the top flange of girder A, the ratio of the moments is smaller than when they were applied at the quarter span section. This is because of the reduction in the girder moments under the application of the horizontal loads and the increase in the moments due to the vertical loads. The ratios of the coefficients at the quarter span and at the midspan sections of girder A when the transverse and the vertical loads are applied at the quarter span of girder A, are approximately 0.391 and 0.515. The ratios are 0.200 and 0.070, respectively when the loads are applied at the midspan of girder A. When the vertical load is applied at the midspan section of girder B the corresponding ratios of the moments for girder B are 0.148 and 0.043.

From Table A16, as from the other tables discussed in this chapter, the remarkable influence of the bar and the plate diaphragms in the behavior of the bridge subjected to the transverse loads is evident. It

was remarked in section 4.3 that the longitudinal, transverse and vertical deflections of the beam diaphragm bridge are of the same order of magnitude as the deflections of the no diaphragm bridge. The behavior, as may be expected, is also reflected in almost equal moments in the two cases. The coefficients for the exterior girders of the beam diaphragm bridge are slightly smaller than the corresponding coefficients of the no diaphragm bridge. Whereas, the coefficients of the interior girders of the beam diaphragm bridge are a little greater than the no diaphragm bridge.

The moment coefficients for the no diaphragm bridge girders and the beam diaphragm bridge girders are as much as ten to fifty times greater than the corresponding coefficients of the bar and the plate diaphragm bridge girders. This comparison of the no diaphragm bridge with the bar diaphragm bridge does not hold when the transverse loads are applied at the bottom flange of girder A. Here, the moments of the bar diaphragm bridge are of comparable magnitudes to the moments of the no diaphragm bridge girders. They are, at places, even larger than the no diaphragm bridge girders.

The moments of the bar diaphragm bridge are invariably smaller than the moments of the plate diaphragm bridge girders. This may be ascribed to the smaller effective stiffness of the plate diaphragms. The differences in the coefficients of the bar diaphragm and the plate diaphragms are maximum when the loads are farthest from the midspan section of girder A. The ratio of the coefficients of the bar diaphragm bridge over the plate diaphragm bridge at quarter span of girder A, when the load is applied at $x=0$ is almost 3. The ratio reduces when the loads move closer to the midspan and

are almost equal when the load is applied at the midspan. However, it is noticed that the distribution of the coefficients from one girder to the other and the distribution along the span is similar in the bar and the plate diaphragm bridge girders.

The coefficients of the composite girder moments are significantly changed when the transverse loads are applied on the bottom flange of girder A instead of at its top flange. The coefficients of the plate diaphragm or the bar diaphragm bridge girders have increased, whereas they have decreased in the simply supported no diaphragm or the beam diaphragm bridge. The quarter span and midspan moments of the plate and the bar diaphragm bridge girder A have increased by a ratio of about five to seven. On the other hand, the factor of reduction for the quarter span moment coefficient of the no diaphragm bridge is about nineteen when the loads are applied at the quarter section. It is about 2.2 for the midspan moment coefficient when the load is applied at the midspan of the bottom flange of girder A. Besides the above changes, another change that may be noted is the lack of symmetry of the moment coefficients about the longitudinal axis of symmetry of the no diaphragm bridge. The distribution of the coefficients was symmetric about this axis for the no diaphragm and the beam diaphragm bridge for top flange loading. The observation is in complete agreement with the deflected shape of the bridge (section 4.3).

In Chapter 3, the remarkable influence of the boundary conditions on the composite girder moments of the no diaphragm bridge were noted. From table VI it was noted that the moments at the quarter span and at the

midspan sections of the symmetrically supported (both ends pinned) no diaphragm bridge are equal to the bending moments of the beam fixed at both ends. A similar influence of the boundary conditions is noted in the distribution of the moments when the transverse loads are applied on the top flange of the no diaphragm bridge. The moment coefficients at the quarter span of girder A are reduced by almost a factor of three when the loads are applied at $x=0$ and at $x=L/4$. The reduction in the moments is much more when the load is applied at the midspan section.

When the transverse load is applied at $x=0$ the three-quarter span moments of the exterior girders are about 10% smaller than the quarter span moments. They are about 25% smaller when the load is applied at $x=L/4$ and are equal when the load is applied at the midspan of girder A. The later trend is entirely opposite for the interior girders. Here, three-quarter span moments are greater than the quarter span moments. The distribution is completely changed when the load is applied at the bottom flange of the girder A instead of on its top flange. The moments of all the girders are greater at the midspan section than at the quarter span or three-quarter span sections.

When the boundary conditions of the bar diaphragm bridge are changed from nonsymmetric to symmetric, the coefficients of the composite moments do not change at the quarter span section of the girders. They become smaller at the midspan section of the bridge. For both the boundary conditions, there is no symmetry of the moments about the longitudinal axis of symmetry of the bridge.

4.6 TRANSVERSE FORCE RESISTED BY DIAPHRAGMS

In order to evaluate the role of diaphragms in transmitting the applied loads to the end supports, it is important to study the forces resisted by the diaphragms:

Tables A17 to A19, in Appendix 2, show the transverse components of forces resisted by various diaphragms for different load positions. Table A17 shows the transverse components of forces resisted by the diaphragms between girders A and B. The other two tables, A18 and A19, show the forces resisted by the diaphragms between girders B and C, and girders C and D respectively.

From these tables it is apparent that when the load is applied at the top flange of girder A the intermediate diaphragms contribute essentially nothing in transmitting the applied load from one girder to the other. The same behavior is also reflected in the bridge deflections in Tables A1 to A12. Here, the deflections of the slab of the bridge with the bar diaphragms provided only at the end sections are seen to be essentially the same as the deflections of the bridge with bar diaphragms at quarter span intervals. The transverse deflections of the bottom flange of the loaded and unloaded girders are somewhat smaller when intermediate diaphragms are missing. The insignificance of the intermediate diaphragms for the top loading of the exterior girder is also reflected in Tables A13 to A15. Here, one sees that the bridge with intermediate diaphragms and the bridge without the intermediate diaphragms have almost equal reactions.

The behavior is confirmed from almost equal coefficients of moments, M_y , for the two bridges in Table A16.

The role of intermediate diaphragms is important when the loads are applied on the bottom flange of girder A. The intermediate diaphragms now help transmit a sizeable portion of the applied load to the unloaded girders. The diaphragms adjacent to the applied load carry a large proportion of the applied load. The behavior, as may be expected, is also reflected in a changed distribution of the deflections, reactions and moments of the bridge.

In the absence of intermediate diaphragms, the transverse deflections of the bottom flange of the loaded girder A are very similar to the deflections of the no diaphragm bridge. The magnitudes of the transverse deflections of the bottom flange of girder A are greater than the corresponding deflections of the bridge with the intermediate diaphragms. The transverse deflections of the bottom flange of unloaded girders B, C and D are a little smaller for the bridge without the intermediate diaphragms. These deflections, however, are negligible in comparison to the no diaphragm bridge. This is expected since the diaphragms at the end sections prevent a rigid body translation of the bridge deck.

The longitudinal deflections of the top flange of girder A have decreased in the absence of intermediate diaphragms. The longitudinal

reactions of the bridge without the intermediate diaphragms are closer in magnitude to the no diaphragm bridge than the reactions of the bridge with intermediate diaphragms. For all positions of the load on the bottom flange of girder A the longitudinal reactions of girder D, invariably are smaller than the reactions of the no diaphragm bridges.

The magnitudes of the transverse reactions of the bridge in the absence of the intermediate diaphragms are in between the magnitudes of the reactions of the other two types of bridges.

The vertical reactions in the absence of the intermediate diaphragms are similar to the no diaphragm bridge.

From Tables A17 to A19 it is observed that the bar or the plate diaphragms at the end sections of the bridge resist essentially all the transverse load applied to the top flange of girder A. The proportion of the load transmitted by the end diaphragm is directly related to the position of the load. When the load is applied at $x=0$, the end diaphragms at $x=0$ carry almost all the transverse load, but when the load is applied at $x=L$ the diaphragms at $x=0$ contribute essentially nothing in resisting the load.

The proportion of the load carried by the diaphragms between girders A and B, B and C, and girders C and D varies with the position of the load. The diaphragms between girders A and B share a large portion (slightly more than one-third) of the total load when the top

flange of girder A is loaded at the end sections. The transverse loads listed in Tables A17 to A19, for the bar diaphragms, are the components of the loads carried by the inclined members. When the top flange of girder A is loaded, the bars sloping away from the loaded girders are in compression. Whereas the members sloping towards this girder are in tension and the horizontal bars of the intermediate diaphragms are mostly unstressed. They, however, are in compression when the bottom flange is loaded.

The transverse load resisted by the beam diaphragms is very small in comparison to the bar and the plate diaphragms. The behavior is as expected since the deflected shape of the bridge with beam diaphragms is similar to the deflected shape of the no diaphragm bridge.

SUMMARY AND CONCLUSIONS

5.1 SUMMARY

The prime objective of the thesis is to study the behavior of a composite I-beam bridge subjected to horizontal loads and to study the effects of diaphragms on the bridge behavior. Since there is no technique available to consider the effects of horizontal loads on the bridge, a finite element method of analysis is developed which treats the composite I-beam bridge as a space structure. The method treats the bridge as an assemblage of plate, beam and bar elements. The slab and the longitudinal girders are treated as an assemblage of rectangular plate elements. The top and bottom flanges of the longitudinal girders are treated as beam elements.

The stiffness properties of the rectangular plate element having six degrees of freedom (three translations and three rotations) are derived in the second chapter and are listed in Appendix 1. The element is believed to be the only one of its kind. The properties of the element are satisfactorily checked for uniform tensile and shearing strains. The convergence properties of the element are examined with a cantilever beam problem.

A computer program is written in Fortran language to solve a large number of simultaneous algebraic equations. The series method of substructures, as explained in Chapter 3, is adopted for the solution of the equations on the Sigma 7 computer.

Although the prime interest of the presentation is the application of horizontal loads on composite I-beam bridges, several bridges are analysed for the vertical loads in order to compare the present model with other techniques. The comparisons made, in general, have proved satisfactory. Some differences which were noted in the comparisons are attributed to the three dimensional behavior of the bridge considered in the present analysis.

In Chapter 4, a 100 foot span, four girder bridge (Figure 7) without diaphragms and with diaphragms is solved. Three kinds of diaphragms are considered. They are beam, bar and plate diaphragms. The diaphragms are provided at the end sections and quarter span intervals. The transverse loads are applied only on the top and bottom flanges of the exterior girder. A substantial influence of bar and plate diaphragms is noted on the behavior of the bridge. In order to study the role of the intermediate diaphragms the bridge is also solved without intermediate diaphragms. From the deflected shape and the reactions of the bridge it is noted that the intermediate diaphragms are useful only when the bottom flange is loaded.

The bridge is solved for two boundary conditions: (1) symmetric, where the ends of the bottom flanges of longitudinal girders are restrained from translating in all directions, (2) nonsymmetric, where the bottom flange is free to translate in the longitudinal direction at one end. All other translations are prevented at both ends.

Finally, as a check on the general conclusions made in Chapter 4 regarding the influence of the diaphragms on the bridge behavior, a four girder bridge, symmetrically supported over a span of 76' is solved. The bridge is not provided with diaphragms in one case and is provided with bar diaphragms at quarter span intervals in the second case. The conclusions made earlier are seen to hold for this bridge.

5.2 CONCLUSIONS

On the basis of the present investigation certain general conclusions may be drawn regarding the finite element method of analysis and the behavior of the bridge under the influence of horizontal loads.

The finite element model developed in the present investigation seems to provide a satisfactory analytical approach to study the effects of horizontal loads on the composite I-beam bridges. The limitations of the orthotropic plate theory and Gustafson's finite element method which stem from the assumptions of plane sections remain plane and from the fact that the three dimensional structure is treated as a two dimensional structure are eliminated in the present technique. The other major advantage of the method over the existing methods is that the influence of various parameters like diaphragms (their type, spacing and size) and the lateral bracing can be examined accurately.

The comparisons made in Chapter 3 for the vertically loaded composite bridges provide a very strong basis for treating a composite I-beam bridge as a three dimensional space structure. It was noted that

the bridge has rather significant longitudinal and transverse reactions when subjected to vertical loads. Since the horizontal reactions occur at the bottom plane they affect the composite girder moments. The moments of the various girders are affected approximately in proportion to magnitudes and elevations of the longitudinal reactions.

The inclusion of the beam diaphragms reduces the composite girder moment carried by the loaded girder. The reduction is greater when the vertical load is applied on the interior girder than when it is applied on the exterior girder.

The influence of the boundary conditions on the composite girder moments is also very pronounced. When the girders are simply supported the total composite moment at any section of the bridge is almost equal to the bending moment in a transversely loaded beam on simple supports. The moments, however, correspond to the bending moments of the fixed beam when longitudinal girders are pinned at both ends.

The no diaphragm bridge undergoes considerable warping when subjected to horizontal loads at the top or bottom flange of the loaded exterior girder. Beam diaphragms with nodes at the middle plane of the slab are essentially no help in preventing the warping of the cross section. For transverse loads the slab of the no diaphragm bridge (or the bridge with the beam diaphragms) translates almost like a rigid body in the transverse direction. The load applied on the exterior girder is transmitted from one girder to the other through large distortions of the

longitudinal girders.

When bar or plate diaphragms are included, the magnitudes of the bridge deflections are considerably reduced. Here, the end diaphragms act as transverse bracing and prevent the rigid body translation of the deck. The cross section of the bridge retains its shape to a large extent.

When the load is applied on the top flange of girder A, the intermediate diaphragms do not seem to serve any purpose in transferring the load. But, when the loads are applied on the bottom flange, these intermediate diaphragms provide an efficient transfer of load from one girder to another and resist excessive transverse deflections of the longitudinal girders.

Significant differences are noted in the reactions of the no diaphragm bridge girders and the reactions of the bar diaphragm bridge girders. The longitudinal reactions of the no diaphragm bridge are much greater than the transverse and vertical reactions. The longitudinal reactions are considerably reduced for the bar diaphragm bridge. They are now comparable in magnitude to the transverse and vertical reactions.

The transverse reactions of the no diaphragm bridge have almost a fixed value for various positions of the load on the top flange of the exterior girder. These reactions vary linearly with load position for the bridge with bar diaphragms. For this bridge the reactions

of the interior girders are almost double the reactions for the exterior girders.

When the loads are applied on the bottom flange of the exterior girder of the no diaphragm bridge, the loaded girder undergoes considerable transverse deflection and resists a sizeable portion of the applied load. The rest of the applied load is carried by the unloaded girders in equal proportion. The behavior of the bridge with the bar diaphragms is similar to the bridge subjected to top loading.

The coefficients of the composite girder moments at various sections of the bar diaphragm bridge are almost negligible in comparison to the corresponding coefficients of the no diaphragm bridge.

The changes in the transverse reactions of the girders, when the boundary conditions of the bridge are changed from nonsymmetric to symmetric are small. For the no diaphragm bridge the distribution of the transverse reactions is a little more uniform for the top loading. There is almost no change in the reactions when the bottom flange of the exterior girder is loaded. When the bar diaphragm bridge is subjected to the top loading, there is very little change in the transverse reactions. But for the case of the bottom loading, there is considerable redistribution of the total reaction resisted by the two ends of a girder. With the above change in the boundary conditions the coefficients of moments are reduced almost by a factor of three for the no diaphragm bridge subjected to the

transverse loads at the top flange of the exterior girder, whereas virtually no change is noted for the bar diaphragm bridge.

5.3 RECOMMENDATIONS FOR FURTHER STUDY

The finite element model developed in the present analysis has been verified with several approximate methods of bridge analysis. The element is also checked to see if it can exactly predict the theory of elasticity solution for uniform tensile and shearing strains. A true representation of these strains is a necessary criterion for convergence of the solution with the reduction of element size. However, an experimental investigation of a model bridge is recommended in order to increase confidence in the analytical approach.

The extension of the method to study various other parameters that affect the behavior of bridges like the relative stiffness of various members, number and spacing of longitudinal girders and diaphragms, are recommended. The computer program in its present form can solve the problem of a bridge on continuous supports.

The extension of the method for the stability analysis of bridge structures also seems to be an interesting field for future work.

The method is capable of solving folded plate and shell problems and hence it opens up another avenue of research.

APPENDIX 1

Stiffness Matrices for Plate and Beam Elements

Dimensionless stiffness matrix $[k]$ for transverse bending of the rectangular plate element of figure 3.

$$\{F\} = [k]\{\delta\}$$

$$\text{where } \{F\} = \begin{pmatrix} F_i \\ F_j \\ F_k \\ F_l \end{pmatrix}$$

$$\{\delta\} = \begin{pmatrix} \delta_i \\ \delta_j \\ \delta_k \\ \delta_l \end{pmatrix}$$

$$\{F_i\} = \begin{pmatrix} F_{wi} \\ F_{\theta xi/b} \\ F_{\theta yi/a} \end{pmatrix}$$

$$\{\delta_i\} = \begin{pmatrix} W_i \\ b\theta_{xi} \\ z\theta_{yi} \end{pmatrix}$$

The complete 12×12 stiffness matrix of a rectangular finite element may be constructed from the 12×3 matrices associated with the displacements of each joint. Thus, the stiffness properties of the element are completely defined by the 12×3 matrix associated with any joint i .

The 12x3 stiffness matrix associated with joint i:

$$\begin{bmatrix}
 12C_1 + 12C_2 + 2C_3 + 42C_4 & 6C_2 + C_3 + 3C_4 & -6C_1 - C_3 - 3C_4 \\
 6C_2 + C_3 + 3C_4 & 4C_2 + 4C_4 & -C_3 \\
 -6C_1 - C_3 - 3C_4 & -C_3 & 4C_1 + 4C_4 \\
 6C_1 - 12C_2 - 2C_3 - 42C_4 & -6C_2 - 3C_4 & -3C_1 + C_3 + 3C_4 \\
 6C_2 + 3C_4 & 2C_2 - C_4 & 0 \\
 -3C_1 + C_3 + 3C_4 & 0 & 2C_1 - 4C_4 \\
 -6C_1 - 6C_2 + 2C_3 + 42C_4 & -3C_2 + 3C_4 & 3C_1 - 3C_4 \\
 3C_2 - 3C_4 & C_2 + C_4 & 0 \\
 -3C_1 + 3C_4 & 0 & C_1 + C_4 \\
 -12C_1 + 6C_2 - 2C_3 - 42C_4 & 3C_2 - C_3 - 3C_4 & 6C_1 + 3C_4 \\
 3C_2 - C_3 - 3C_4 & 2C_2 - 4C_4 & 0 \\
 -6C_1 - 3C_4 & 0 & 2C_1 - C_4
 \end{bmatrix}$$

Where $C_1 = D(b/a)^2 / (3ab),$

$C_2 = D(a/b)^2 / (3ab),$

$C_3 = \mu D / (ab),$

$C_4 = D(1-\mu) / (15ab),$

and $D = Et^3 / 12(1-\mu^2)$

Dimensionless stiffness matrix for the plane stress rectangular plate element of figure 3.

$$\{F\} = [k]\{\delta\}$$

where

$$\{F\} = \begin{Bmatrix} F_i \\ F_j \\ F_k \\ F_l \end{Bmatrix}, \quad \{F_i\} = \begin{Bmatrix} bF_{ui} \\ aF_{vi} \\ F_{\theta zi} \end{Bmatrix}$$

$$\{\delta\} = \begin{Bmatrix} \delta_i \\ \delta_j \\ \delta_k \\ \delta_l \end{Bmatrix}; \quad \{\delta_i\} = \begin{Bmatrix} U_{i/b} \\ V_{i/a} \\ \theta_{zi} \end{Bmatrix}$$

$$\text{Let } [k] = \frac{1}{120} \left[\left(\frac{a}{b}\right)^2 [k_1] + \left(\frac{b}{a}\right)^2 [k_2] + \gamma [k_3] + \left(\frac{1-\gamma}{2}\right) [k_4] \right]$$

where $[k_1]$, $[k_2]$, $[k_3]$ and $[k_4]$ are as defined in the following pages.

Matrix [C] relating dimensionless degrees of freedom $\{\delta\}$ with the generalized coefficients $\{a\}$.

1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	-1/2	1/2	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	1	0
0	1	0	1	0	0	0	0	0	0	0	1	0	0	0	0	0	1	0	0
0	0	0	0	-1/2	1/2	0	1/2	-1	0	0	0	0	1/2	0	0	0	0	-3/2	0
1	0	1	0	1	0	1	0	1	0	1	0	1	0	1	0	1	0	1	0
0	1	0	1	0	1	0	1	0	1	0	1	0	1	0	1	0	1	0	1
0	0	0	0	-1/2	1/2	-1/2	1/2	-1	1	0	0	-1/2	1/2	-1	1	0	0	-3/2	3/2
1	0	1	0	0	0	0	0	0	0	1	0	0	0	0	0	1	0	0	0
0	1	0	0	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	1
0	0	0	0	-1/2	1/2	-1/2	0	0	1	0	0	-1/2	0	0	0	0	0	0	3/2

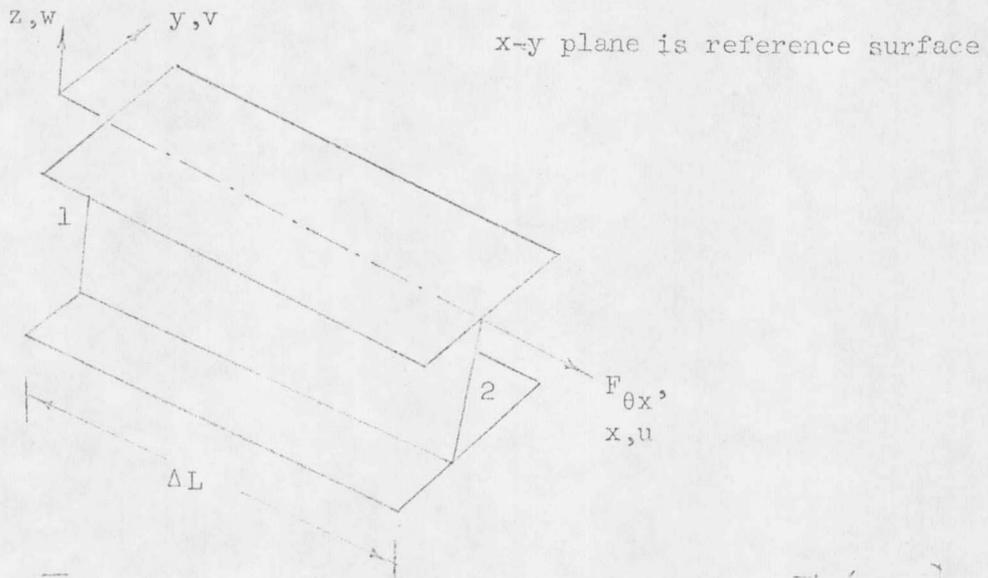
Column number

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

Column number

1 2 3 4 5 6 7 8 9 10 11 12 13 14

Stiffness Matrix for the beam element



F_{v1}	$\frac{A_s}{\Delta L}$					v_1			
F_{w1}	0	$\frac{12J_x}{\Delta L^3}$				w_1			
$F_{\theta x1}$	$-\frac{S_x}{\Delta L}$	$-\frac{6J_x}{\Delta L^2}$	$\frac{4J_x}{\Delta L}$			θ_{x1}			
$F_{\theta y1}$	0	0	0	γ					
F_{v2}	$-\frac{A_s}{\Delta L}$	0	$\frac{S_x}{\Delta L}$	0	$\frac{A_s}{\Delta L}$	y_2			
F_{w2}	0	$-\frac{12J_x}{\Delta L^3}$	$\frac{6J_x}{\Delta L^2}$	0	0	w_2			
$F_{\theta x2}$	$\frac{S_x}{\Delta L}$	$-\frac{6J_x}{\Delta L^2}$	$\frac{2J_x}{\Delta L}$	0	$-\frac{S_x}{\Delta L}$	$\frac{6J_x}{\Delta L^2}$	$\frac{J_x}{\Delta L}$	θ_{x2}	
$F_{\theta y2}$	0	0	0	$-\gamma$	0	0	0	γ	θ_{y2}

Symmetric

APPENDIX 2

Bridge Deflections, Reactions and Composite
Girder Moments and Diaphragm Forces

TABLE A1 LONGITUDINAL DEFLECTIONS U FOR GIRDER A IN MICROINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.9888	-0.5282	-0.1423	0.2024	0.5265	0.0391	0.1152	0.1201	
1L/4	-0.9358	-0.5212	-0.1333	0.2128	0.5388	0.0293	0.1046	0.1145	U
2L/4	-0.8707	-0.5102	-0.1365	0.2338	0.5841	0.0003	0.0660	0.0905	NN6
3L/4	-0.8120	-0.4826	-0.1398	0.2380	0.6357	-0.0224	0.0282	0.0614	L
4L/4	-0.7834	-0.4639	-0.1309	0.2368	0.6724	-0.0275	0.0170	0.0501	
0L/4	-0.9828	-0.5257	-0.1402	0.2041	0.5281	0.0340	0.1087	0.1159	
1L/4	-0.9328	-0.5192	-0.1315	0.2143	0.5400	0.0260	0.0999	0.1112	U
2L/4	-0.8688	-0.5088	-0.1353	0.2350	0.5851	0.0006	0.0660	0.0902	NB6
3L/4	-0.8105	-0.4815	-0.1390	0.2388	0.6358	-0.0192	0.0327	0.0646	L
4L/4	-0.7823	-0.4633	-0.1304	0.2370	0.6695	-0.0229	0.0241	0.0557	
0L/4	-0.0443	0.0496	0.0773	0.0665	0.0356	0.1428	0.1775	0.1274	
1L/4	-0.0371	0.0206	0.0576	0.0562	0.0349	0.0827	0.1437	0.1108	U
2L/4	-0.0354	-0.0268	0.0002	0.0270	0.0345	-0.0198	0.0221	0.0530	NA6
3L/4	-0.0351	-0.0557	-0.0572	-0.0208	0.0358	-0.0780	-0.1007	-0.0506	L
4L/4	-0.0351	-0.0657	-0.0770	-0.0503	0.0423	-0.0978	-0.1399	-0.1156	
0L/4	-0.0944	0.0253	0.0772	0.0907	0.0841	0.1410	0.1977	0.1624	
1L/4	-0.0850	-0.0031	0.0578	0.0799	0.0823	0.0766	0.1641	0.1458	U
2L/4	-0.0824	-0.0502	0.0005	0.0506	0.0814	-0.0300	0.0393	0.0889	NP6
3L/4	-0.0819	-0.0788	-0.0569	0.0027	0.0827	-0.0872	-0.0862	-0.0181	L
4L/4	-0.0818	-0.0886	-0.0764	-0.0266	0.0903	-0.1062	-0.1248	-0.0877	

TABLE A1 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT.	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.3014	-0.1162	-0.0019			0.0272	0.0483		U SN6 L
1L/4	-0.2589	-0.1154	0.0049			0.0174	0.0385		
2L/4	-0.2023	-0.1095	0.0000			-0.0118	0.0000		
3L/4	-0.1520	-0.0869	-0.0049			-0.0353	-0.0385		
4L/4	-0.1337	-0.0744	0.0019			-0.0400	-0.0483		
0L/4	-0.0381	0.0526	0.0770			0.1355	0.1565		U SA6 L
1L/4	-0.0305	0.0238	0.0574			0.0748	0.1213		
2L/4	-0.0289	-0.0237	0.0000			-0.0275	0.0000		
3L/4	-0.0292	-0.0528	-0.0574			-0.0851	-0.1213		
4L/4	-0.0304	-0.0633	-0.0770			-0.1034	-0.1565		
0L/4	-0.0384	0.0530	0.0771			0.0816	0.1116		U SA6* L
1L/4	-0.0307	0.0239	0.0575			0.0562	0.0806		
2L/4	-0.0289	-0.0238	0.0000			-0.0036	0.0000		
3L/4	-0.0292	-0.0528	-0.0575			-0.0574	-0.0806		
4L/4	-0.0304	-0.0633	-0.0771			-0.0769	-0.1116		
0L/4	-0.2452	-0.0878	0.0114			0.0322	0.0638		U SN4 L
1L/4	-0.2125	-0.0894	0.0145			0.0177	0.0508		
2L/4	-0.1797	-0.0960	0.0000			-0.0223	0.0000		
3L/4	-0.1532	-0.0899	-0.0145			-0.0519	-0.0508		
4L/4	-0.1426	-0.0832	-0.0114			-0.0571	-0.0638		
0L/4	-0.0373	0.0358	0.0565			0.1045	0.1178		U SA4 L
1L/4	-0.0295	0.0146	0.0423			0.0569	0.0924		
2L/4	-0.0268	-0.0203	0.0000			-0.0220	0.0000		
3L/4	-0.0269	-0.0416	-0.0423			-0.0645	-0.0924		
4L/4	-0.0279	-0.0494	-0.0565			-0.0779	-0.1178		

TABLE A1 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.5240	-0.2825	-0.0478	0.1869	0.4224	-0.0710	-0.0674	-0.0229	U
2L/4	-0.9398	-0.5330	-0.1359	0.2578	0.6540	-0.0022	0.0489	0.0687	NN6
3L/4	-1.2240	-0.7205	-0.2255	0.2629	0.7520	0.0800	0.2017	0.1988	L
4L/4	-1.3455	-0.8088	-0.2800	0.2424	0.7618	0.1060	0.2596	0.2574	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.5233	-0.2838	-0.0505	0.1850	0.4203	-0.0310	-0.0086	0.0199	U
2L/4	-0.9372	-0.5310	-0.1353	0.2571	0.6533	0.0049	0.0634	0.0822	NB6
3L/4	-1.2184	-0.7160	-0.2202	0.2671	0.7538	0.0387	0.1417	0.1556	L
4L/4	-1.3367	-0.8013	-0.2719	0.2519	0.7685	0.0432	0.1640	0.1848	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0291	-0.0136	-0.0020	0.0102	0.0232	-0.0519	-0.1086	-0.0621	U
2L/4	-0.0410	-0.0199	-0.0001	0.0198	0.0402	0.0285	-0.0015	-0.0317	NA6
3L/4	-0.0480	-0.0228	0.0024	0.0264	0.0524	0.0594	0.1534	0.1225	L
4L/4	-0.0507	-0.0249	0.0014	0.0253	0.0510	0.0671	0.1940	0.2383	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0589	-0.0286	-0.0021	0.0255	0.0537	-0.0715	-0.1333	-0.0629	U
2L/4	-0.0927	-0.0449	0.0003	0.0452	0.0918	0.0349	0.0118	-0.0196	NP6
3L/4	-0.1122	-0.0544	0.0029	0.0583	0.1155	0.0657	0.2134	0.1894	L
4L/4	-0.1192	-0.0590	0.0014	0.0597	0.1180	0.0697	0.2636	0.3496	

TABLE A1 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.1832	-0.0783	0.0215			-0.0642	-0.0810		U
2L/4	-0.2722	-0.1329	0.0000			-0.0034	0.0000		SN6
3L/4	-0.2276	-0.1231	-0.0215			0.0475	0.0810		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0351	-0.0165	-0.0016			-0.0436	-0.0857		U
2L/4	-0.0408	-0.0198	0.0000			0.0291	0.0000		SA6
3L/4	-0.0292	-0.0136	0.0016			0.0355	0.0857		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0344	-0.0166	-0.0026			-0.0699	-0.1003		U
2L/4	-0.0416	-0.0189	0.0000			0.0076	0.0000		SA6*
3L/4	-0.0300	-0.0133	0.0026			0.0697	0.1003		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.1787	-0.0810	0.0111			-0.0882	-0.1036		U
2L/4	-0.2503	-0.1221	0.0000			-0.0084	0.0000		SN4
3L/4	-0.1998	-0.1040	-0.0111			0.0531	0.1036		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0341	-0.0146	0.0002			-0.0424	-0.0824		U
2L/4	-0.0389	-0.0194	0.0000			0.0283	-0.0000		SA4
3L/4	-0.0276	-0.0139	-0.0002			-0.0327	0.0824		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A2 LONGITUDINAL DEFLECTIONS U FOR GIRDER B IN MICROINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF									BRIDGE TYPE
	TOP FLANGE AT X=					BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4		
0L/4	-0.3095	-0.1782	-0.0473	0.0680	0.1764	-0.0008	0.0224	0.0305		
1L/4	-0.3073	-0.1735	-0.0466	0.0697	0.1787	-0.0001	0.0211	0.0289	U	
2L/4	-0.2884	-0.1675	-0.0454	0.0755	0.1931	-0.0001	0.0195	0.0267	NN6	
3L/4	-0.2691	-0.1592	-0.0440	0.0795	0.2075	-0.0025	0.0167	0.0254	L	
4L/4	-0.2611	-0.1546	-0.0433	0.0813	0.2041	-0.0031	0.0163	0.0263		
0L/4	-0.3132	-0.1784	-0.0476	0.0676	0.1759	0.0049	0.0294	0.0349		
1L/4	-0.3075	-0.1733	-0.0465	0.0698	0.1787	0.0044	0.0273	0.0331	U	
2L/4	-0.2879	-0.1671	-0.0451	0.0757	0.1933	0.0010	0.0215	0.0285	NB6	
3L/4	-0.2686	-0.1588	-0.0437	0.0797	0.2084	-0.0035	0.0151	0.0242	L	
4L/4	-0.2601	-0.1537	-0.0425	0.0820	0.2084	-0.0049	0.0130	0.0233		
0L/4	-0.0012	0.0116	0.0233	0.0209	0.0117	0.0383	0.0578	0.0426		
1L/4	-0.0105	0.0058	0.0166	0.0176	0.0114	0.0252	0.0446	0.0368	U	
2L/4	-0.0119	-0.0074	0.0000	0.0074	0.0111	-0.0041	0.0074	0.0149	NA6	
3L/4	-0.0119	-0.0176	-0.0165	-0.0059	0.0094	-0.0257	-0.0305	-0.0163	L	
4L/4	-0.0119	-0.0207	-0.0232	-0.0117	-0.0002	-0.0323	-0.0455	-0.0314		
0L/4	-0.0168	0.0032	0.0233	0.0289	0.0278	0.0254	0.0602	0.0538		
1L/4	-0.0261	-0.0021	0.0166	0.0255	0.0272	0.0187	0.0453	0.0468	U	
2L/4	-0.0273	-0.0151	0.0001	0.0151	0.0268	-0.0052	0.0120	0.0225	NP6	
3L/4	-0.0272	-0.0252	-0.0163	0.0019	0.0249	-0.0286	-0.0228	-0.0049	L	
4L/4	-0.0271	-0.0283	-0.0230	-0.0037	0.0150	-0.0359	-0.0392	-0.0136		

TABLE A2 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.0802	-0.0406	-0.0004			-0.0043	0.0008		
1L/4	-0.0816	-0.0382	-0.0004			-0.0033	0.0003		U
2L/4	-0.0658	-0.0340	0.0000			-0.0025	0.0000		SN6
3L/4	-0.0497	-0.0278	0.0004			-0.0035	-0.0003		L
4L/4	-0.0454	-0.0254	0.0004			-0.0044	-0.0008		
0L/4	0.0012	0.0128	0.0233			0.0359	0.0504		
1L/4	-0.0080	0.0070	0.0166			0.0227	0.0369		U
2L/4	-0.0094	-0.0062	0.0000			-0.0066	0.0000		SA6
3L/4	-0.0096	-0.0165	-0.0166			-0.0279	-0.0369		L
4L/4	-0.0099	-0.0198	-0.0233			-0.0340	-0.0504		
0L/4	0.0016	0.0123	0.0233			0.0122	0.0197		
1L/4	-0.0078	0.0069	0.0164			0.0092	0.0139		U
2L/4	-0.0095	-0.0061	0.0000			0.0002	0.0000		SA6*
3L/4	-0.0096	-0.0165	-0.0164			-0.0106	-0.0139		L
4L/4	-0.0100	-0.0198	-0.0233			-0.0152	-0.0197		
0L/4	-0.0585	-0.0328	0.0037			-0.0116	-0.0016		
1L/4	-0.0635	-0.0301	0.0014			-0.0080	-0.0018		U
2L/4	-0.0580	-0.0286	0.0000			-0.0042	0.0000		SN4
3L/4	-0.0497	-0.0282	-0.0014			-0.0055	0.0018		L
4L/4	-0.0485	-0.0284	-0.0037			-0.0073	0.0016		
0L/4	0.0029	0.0057	0.0160			0.0226	0.0371		
1L/4	-0.0054	0.0038	0.0107			0.0157	0.0263		U
2L/4	-0.0085	-0.0044	-0.0000			-0.0041	-0.0000		SA4
3L/4	-0.0088	-0.0125	-0.0107			-0.0210	-0.0263		L
4L/4	-0.0091	-0.0150	-0.0160			-0.0256	-0.0371		

TABLE A2 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.1726	-0.0984	-0.0217	0.0557	0.1329	0.0623	0.1054	0.0844	U
2L/4	-0.3102	-0.1786	-0.0471	0.0833	0.2122	0.0016	0.0389	0.0527	NN6
3L/4	-0.4001	-0.2337	-0.0685	0.0938	0.2502	-0.0674	-0.0697	-0.0305	L
4L/4	-0.4257	-0.2477	-0.0711	0.1023	0.2682	-0.0924	-0.1163	-0.0758	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.1697	-0.0928	-0.0143	0.0622	0.1397	0.0117	0.0307	0.0301	U
2L/4	-0.3098	-0.1779	-0.0452	0.0866	0.2153	-0.0030	0.0242	0.0364	NB6
3L/4	-0.4056	-0.2391	-0.0757	0.0874	0.2468	-0.0116	0.0112	0.0272	L
4L/4	-0.4385	-0.2604	-0.0852	0.0858	0.2550	-0.0124	0.0078	0.0199	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0039	-0.0051	-0.0029	0.0022	0.0076	-0.0136	-0.0119	-0.0118	U
2L/4	-0.0119	-0.0052	-0.0002	0.0047	0.0117	-0.0109	0.0007	0.0128	NA6
3L/4	-0.0161	-0.0066	0.0031	0.0096	0.0107	0.0122	0.0259	0.0330	L
4L/4	-0.0174	-0.0075	0.0031	0.0129	0.0094	0.0201	0.0471	0.0312	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0137	-0.0100	-0.0024	0.0073	0.0174	-0.0003	0.0362	0.0210	U
2L/4	-0.0280	-0.0141	-0.0002	0.0137	0.0281	-0.0377	0.0116	0.0576	NP6
3L/4	-0.0367	-0.0172	0.0027	0.0202	0.0318	-0.0130	-0.0144	0.0224	L
4L/4	-0.0395	-0.0184	0.0034	0.0238	0.0335	-0.0003	0.0019	-0.0341	

TABLE A2 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF								BRIDGE TYPE
	TOP FLANGE AT X=				BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0590	-0.0300	0.0023			0.0473	0.0741		U
2L/4	-0.0859	-0.0434	0.0000			-0.0132	0.0000		SN6
3L/4	-0.0691	-0.0358	-0.0023			-0.0554	-0.0741		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0057	-0.0060	-0.0027			-0.0109	-0.0050		U
2L/4	-0.0115	-0.0050	0.0000			-0.0107	0.0000		SA6
3L/4	-0.0094	-0.0034	0.0027			0.0046	0.0050		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0064	-0.0058	-0.0016			0.00480	0.0712		U
2L/4	-0.0105	-0.0060	0.0000			-0.0095	0.0000		SA6*
3L/4	-0.0085	-0.0038	0.0016			-0.0505	-0.0712		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0530	-0.0286	0.0010			0.0529	0.0880		U
2L/4	-0.0769	-0.0398	0.0000			-0.0248	0.0000		SN4
3L/4	-0.0612	-0.0316	-0.0010			-0.0673	-0.0880		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0037	-0.0062	-0.0033			-0.0082	0.0004		U
2L/4	-0.0100	-0.0043	0.0000			-0.0135	-0.0000		SA4
3L/4	-0.0087	-0.0029	0.0033			0.0026	-0.0004		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A3 LONGITUDINAL DEFLECTIONS U FOR GIRDER C IN MICROINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF								BRIDGE TYPE
	TOP FLANGE AT X=					BOTTOM FLANGE AT X=			
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.3265	0.1724	0.0453	-0.0689	-0.1763	-0.0159	-0.0424	-0.0428	
1L/4	0.3106	0.1722	0.0436	-0.0707	-0.1785	-0.0113	-0.0369	-0.0396	U
2L/4	0.2882	0.1690	0.0452	-0.0775	-0.1931	-0.0001	-0.0218	-0.0300	NN6
3L/4	0.2689	0.1597	0.0466	-0.0786	-0.2111	0.0091	-0.0069	-0.0186	L
4L/4	0.2609	0.1551	0.0449	-0.0760	-0.2212	0.0126	-0.0005	-0.0129	
0L/4	0.3250	0.1728	0.0456	-0.0685	-0.1759	-0.0145	-0.0401	-0.0411	
1L/4	0.3096	0.1719	0.0435	-0.0708	-0.1786	-0.0104	-0.0356	-0.0385	U
2L/4	0.2878	0.1687	0.0449	-0.0777	-0.1933	-0.0002	-0.0221	-0.0302	NB6
3L/4	0.2685	0.1594	0.0462	-0.0789	-0.2108	0.0077	-0.0089	-0.0200	L
4L/4	0.2600	0.1542	0.0441	-0.0769	-0.2204	0.0100	-0.0043	-0.0155	
0L/4	0.0152	-0.0166	-0.0253	-0.0219	-0.0119	-0.0485	-0.0587	-0.0419	
1L/4	0.0126	-0.0071	-0.0194	-0.0186	-0.0117	-0.0288	-0.0483	-0.0367	U
2L/4	0.0111	0.0087	-0.0003	-0.0092	-0.0116	0.0050	-0.0086	-0.0183	NA6
3L/4	0.0111	0.0181	0.0188	0.0068	-0.0129	0.0238	0.0312	0.0154	L
4L/4	0.0111	0.0212	0.0248	0.0165	-0.0153	0.0298	0.0432	0.0365	
0L/4	0.0322	-0.0086	-0.0253	-0.0299	-0.0278	-0.0512	-0.0675	-0.0539	
1L/4	0.0288	0.0008	-0.0194	-0.0265	-0.0272	-0.0284	-0.0571	-0.0492	U
2L/4	0.0270	0.0165	-0.0003	-0.0170	-0.0269	0.0084	-0.0150	-0.0313	NP6
3L/4	0.0268	0.0257	0.0188	-0.0011	-0.0283	0.0267	0.0269	0.0051	L
4L/4	0.0268	0.0288	0.0246	0.0087	-0.0311	0.0322	0.0390	0.0295	

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TABLE A3 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT:	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0972	0.0351	-0.0014			-0.0124	-0.0207		U SN6 L
1L/4	0.0849	0.0370	-0.0023			-0.0076	-0.0153		
2L/4	0.0656	0.0358	0.0000			0.0038	0.0000		
3L/4	0.0495	0.0286	0.0023			0.0135	0.0153		
4L/4	0.0452	0.0261	0.0014			0.0165	0.0207		
0L/4	0.0133	-0.0175	-0.0250			-0.0452	-0.0505		U SA6 L
1L/4	0.0106	-0.0080	-0.0191			-0.0254	-0.0396		
2L/4	0.0092	0.0078	0.0000			0.0084	0.0000		
3L/4	0.0093	0.0172	0.0191			0.0270	0.0396		
4L/4	0.0096	0.0205	0.0250			0.0326	0.0505		
0L/4	0.0136	-0.0178	-0.0251			-0.0288	-0.0396		U SA6* L
1L/4	-0.0107	-0.0081	-0.0192			-0.0201	-0.0288		
2L/4	0.0091	0.0079	0.0000			0.0012	0.0000		
3L/4	0.0092	0.0173	0.0192			0.0205	0.0288		
4L/4	-0.0096	0.0206	0.0251			0.0274	0.0396		
0L/4	0.0777	0.0243	-0.0066			-0.0153	-0.0279		U SN4 L
1L/4	0.0707	0.0284	-0.0057			-0.0085	-0.0203		
2L/4	0.0579	0.0316	-0.0000			0.0073	-0.0000		
3L/4	0.0493	0.0293	0.0057			0.0196	0.0203		
4L/4	0.0482	0.0294	0.0066			0.0236	0.0279		
0L/4	0.0123	-0.0128	-0.0186			-0.0359	-0.0379		U SA4 L
1L/4	0.0109	-0.0052	-0.0146			-0.0198	-0.0304		
2L/4	0.0084	0.0070	-0.0000			0.0068	-0.0000		
3L/4	0.0084	0.0136	0.0146			0.0202	0.0304		
4L/4	0.0087	0.0160	0.0186			0.0241	0.0379		

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TABLE A3 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BRIDGE BOTTOM FLANGE AT X= TYPE		
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1L/4	0.1768	0.0978	0.0215	-0.0558	-0.1342	0.0317	0.0335	0.0162 U
2L/4	0.3140	0.1779	0.0468	-0.0832	-0.2156	0.0084	-0.0096	-0.0211 NN6
3L/4	0.4025	0.2333	0.0682	-0.0936	-0.2560	-0.0340	-0.0777	-0.0746 L
4L/4	0.4276	0.2474	0.0711	-0.1024	-0.2736	-0.0581	-0.1155	-0.1034
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1L/4	0.1720	0.0930	0.0168	-0.0612	-0.1399	0.0137	0.0078	-0.0028 U
2L/4	0.3114	0.1755	0.0451	-0.0842	-0.2165	-0.0006	-0.0196	-0.0262 NB6
3L/4	0.4060	0.2378	0.0727	-0.0883	-0.2497	-0.0168	-0.0525	-0.0557 L
4L/4	0.4385	0.2597	0.0828	-0.0909	-0.2605	-0.0241	-0.0687	-0.0715
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1L/4	0.0089	0.0042	0.0011	-0.0030	-0.0077	0.0180	0.0401	0.0237 U
2L/4	0.0133	0.0060	-0.0002	-0.0063	-0.0137	-0.0133	-0.0002	0.0134 NA6
3L/4	0.0154	0.0068	-0.0016	-0.0088	-0.0176	-0.0241	-0.0576	-0.0444 L
4L/4	0.0161	0.0073	-0.0018	-0.0095	-0.0184	-0.0259	-0.0729	-0.0885
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1L/4	0.0188	0.0090	0.0011	-0.0081	-0.0179	0.0304	0.0577	0.0334 U
2L/4	0.0310	0.0141	-0.0002	-0.0145	-0.0308	-0.0143	-0.0022	0.0118 NP6
3L/4	0.0374	0.0174	-0.0018	-0.0194	-0.0383	-0.0338	-0.0885	-0.0767 L
4L/4	0.0395	0.0185	-0.0022	-0.0217	-0.0405	-0.0368	-0.1170	-0.1401

TABLE A3 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SN6 L
1L/4	0.0636	0.0295	-0.0023			0.0224	0.0275		
2L/4	0.0899	0.0431	0.0000			0.0040	0.0000		
3L/4	0.0712	0.0357	0.0023			-0.0164	-0.0275		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SA6 L
1L/4	0.0110	0.0052	0.0010			0.0155	0.0328		
2L/4	0.0135	0.0060	0.0000			-0.0129	0.0000		
3L/4	0.0095	0.0040	-0.0010			-0.0149	-0.0328		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SA6* L
1L/4	0.0104	0.0054	0.0016			0.0219	0.0307		
2L/4	0.0139	0.0056	0.0000			0.0004	0.0000		
3L/4	0.0102	0.0037	-0.0016			-0.0214	-0.0307		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SN4 L
1L/4	0.0594	0.0279	-0.0011			0.0271	0.0301		
2L/4	0.0829	0.0393	-0.0000			0.0083	-0.0000		
3L/4	0.0644	0.0316	0.0011			-0.0153	-0.0301		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SA4 L
1L/4	0.0109	0.0047	0.0006			0.0158	0.0327		
2L/4	0.0131	0.0058	-0.0000			-0.0126	-0.0000		
3L/4	0.0090	0.0040	-0.0006			-0.0146	-0.0327		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A4 LONGITUDINAL DEFLECTIONS U FOR GIRDER D IN MICROINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.9682	0.5279	0.1404	-0.2034	-0.5262	-0.0213	-0.0933	-0.1064	
1L/4	0.9324	0.5201	0.1323	-0.2138	-0.5385	-0.0168	-0.0873	-0.1027	U
2L/4	0.8704	0.5098	0.1363	-0.2339	-0.5839	0.0028	-0.0600	-0.0851	NN6
3L/4	0.8117	0.4831	0.1404	-0.2374	-0.6323	0.0199	-0.0323	-0.0643	L
4L/4	0.7830	0.4645	0.1323	-0.2370	-0.6517	0.0220	-0.0270	-0.0593	
0L/4	0.9686	0.5258	0.1386	-0.2051	-0.5279	-0.0295	-0.1046	-0.1141	
1L/4	0.9307	0.5185	0.1308	-0.2151	-0.5398	-0.0233	-0.0965	-0.1092	U
2L/4	0.8686	0.5083	0.1351	-0.2348	-0.5848	-0.0005	-0.0648	-0.0885	NB6
3L/4	0.8103	0.4820	0.1394	-0.2382	-0.6335	0.0195	-0.0326	-0.0644	L
4L/4	0.7821	0.4638	0.1318	-0.2374	-0.6551	0.0234	-0.0244	-0.0571	
0L/4	0.0262	-0.0506	-0.0792	-0.0675	-0.0357	-0.1344	-0.1772	-0.1283	
1L/4	0.0336	-0.0219	-0.0589	-0.0571	-0.0349	-0.0813	-0.1411	-0.1111	U
2L/4	0.0343	0.0265	-0.0005	-0.0273	-0.0344	0.0164	-0.0227	-0.0506	NA6
3L/4	0.0342	0.0560	0.0579	0.0214	-0.0330	0.0769	0.0974	0.0499	L
4L/4	0.0341	0.0660	0.0782	0.0506	-0.0248	0.0972	0.1391	0.1086	
0L/4	0.0751	-0.0259	-0.0791	-0.0917	-0.0840	-0.1186	-0.1912	-0.1620	
1L/4	0.0817	0.0019	-0.0590	-0.0809	-0.0822	-0.0702	-0.1543	-0.1435	U
2L/4	0.0819	0.0498	-0.0007	-0.0508	-0.0812	0.0236	-0.0388	-0.0816	NP6
3L/4	0.0813	0.0792	0.0575	-0.0021	-0.0795	0.0848	0.0786	0.0158	L
4L/4	0.0812	0.0891	0.0778	0.0267	-0.0712	0.1053	0.1205	0.0697	

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TABLE A4 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.2808	0.1161	0.0003			-0.0124	-0.0308		U SN6 L
1L/4	0.2556	0.1146	-0.0057			-0.0082	-0.0260		
2L/4	0.2021	0.1094	0.0000			0.0110	0.0000		
3L/4	0.1518	0.0876	0.0057			0.0273	0.0260		
4L/4	0.1335	0.0752	-0.0003			0.0295	0.0308		
0L/4	0.0205	-0.0533	-0.0787			-0.1262	-0.1552		U SA6 L
1L/4	0.0276	-0.0248	-0.0584			-0.0725	-0.1178		
2L/4	0.0284	0.0237	0.0000			0.0249	0.0000		
3L/4	0.0288	0.0535	0.0584			0.0847	0.1178		
4L/4	0.0299	0.0640	0.0787			0.1034	0.1552		
0L/4	0.0202	-0.0531	-0.0787			-0.0669	-0.0944		U SA6* L
1L/4	0.0275	-0.0247	-0.0583			-0.0470	-0.0683		
2L/4	0.0285	0.0236	0.0000			0.0027	0.0000		
3L/4	0.0288	0.0535	0.0583			0.0494	0.0683		
4L/4	0.0299	0.0641	0.0787			0.0667	0.0944		
0L/4	0.2205	0.0887	-0.0131			-0.0085	-0.0385		U SN4 L
1L/4	0.2057	0.0885	-0.0149			-0.0040	-0.0332		
2L/4	0.1791	0.0953	-0.0000			0.0205	-0.0000		
3L/4	0.1529	0.0907	0.0149			0.0412	0.0332		
4L/4	0.1422	0.0842	0.0131			0.0436	0.0385		
0L/4	0.0174	-0.0361	-0.0583			-0.0916	-0.1157		U SA4 L
1L/4	0.0239	-0.0159	-0.0430			-0.0532	-0.0878		
2L/4	0.0259	0.0198	-0.0000			0.0189	-0.0000		
3L/4	0.0262	0.0423	0.0430			0.0642	0.0878		
4L/4	0.0271	0.0502	0.0583			0.0782	0.1157		

TABLE A4 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	0.5196	0.2827	0.0480	-0.1868	-0.4211	-0.0221	-0.0705	-0.0770	U
2L/4	0.9359	0.5327	0.1357	-0.2579	-0.6506	-0.0051	-0.0748	-0.0981	NN6
3L/4	1.2214	0.7200	0.2248	-0.2636	-0.7462	0.0258	-0.0482	-0.0894	L
4L/4	1.3435	0.8082	0.2792	-0.2433	-0.7565	0.0497	-0.0203	-0.0728	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	0.5209	0.2833	0.0480	-0.1860	-0.4201	0.0058	-0.0297	-0.0471	U
2L/4	0.9355	0.5326	0.1350	-0.2594	-0.6521	-0.0007	-0.0673	-0.0920	NB6
3L/4	1.2179	0.7165	0.2224	-0.2665	-0.7509	-0.0094	-0.0991	-0.1262	L
4L/4	1.3367	0.8012	0.2735	-0.2476	-0.7630	-0.0056	-0.1015	-0.1321	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	0.0219	0.0140	0.0039	-0.0093	-0.0231	0.0461	0.0804	0.0503	U
2L/4	0.0374	0.0180	-0.0001	-0.0182	-0.0382	-0.0077	-0.0010	0.0055	NA6
3L/4	0.0467	0.0214	-0.0049	-0.0278	-0.0455	-0.0509	-0.1255	-0.1130	L
4L/4	0.0499	0.0240	-0.0038	-0.0298	-0.0441	-0.0647	-0.1719	-0.1845	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	0.0529	0.0291	0.0034	-0.0247	-0.0533	0.0394	0.0396	0.0084	U
2L/4	0.0889	0.0439	-0.0004	-0.0444	-0.0890	0.0121	-0.0240	-0.0497	NP6
3L/4	0.1107	0.0532	-0.0048	-0.0596	-0.1089	-0.0239	-0.1161	-0.1381	L
4L/4	0.1185	0.0580	-0.0036	-0.0628	-0.1118	-0.0376	-0.1538	-0.1801	

TABLE A4 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF								BRIDGE TYPE
	TOP FLANGE AT X=				BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	0.1785	0.0785	-0.0213			-0.0057	-0.0212		U
2L/4	0.2681	0.1329	0.0000			0.0127	0.0000		SN6
3L/4	0.2255	0.1229	0.0213			0.0248	0.0212		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	0.0282	0.0170	0.0035			0.0383	0.0588		U
2L/4	0.0377	0.0181	0.0000			-0.0073	0.0000		SA6
3L/4	0.0285	0.0127	-0.0035			-0.0261	-0.0588		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	0.0288	0.0169	0.0028			0.0000	-0.0018		U
2L/4	0.0371	0.0188	0.0000			0.0017	0.0000		SA6*
3L/4	0.0278	0.0131	-0.0028			0.0026	0.0018		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	0.1720	0.0813	-0.0107			0.0080	-0.0154		U
2L/4	0.2441	0.1220	-0.0000			0.0256	-0.0000		SN4
3L/4	0.1965	0.1037	0.0107			0.0305	0.0154		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	0.0248	0.0158	0.0029			0.0342	0.0502		U
2L/4	0.0345	0.0171	-0.0000			-0.0036	-0.0000		SA4
3L/4	0.0267	0.0124	-0.0029			-0.0213	-0.0502		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A5 TRANSVERSE DEFLECTIONS V FOR GIRDER A IN MILLIINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					NODES OF BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.4243	-0.4222	-0.4203	-0.4186	-0.4170	-0.2429	-0.3363	-0.2425	
1L/4	-0.4222	-0.4212	-0.4200	-0.4190	-0.4180	-0.2430	-0.3365	-0.2427	U
2L/4	-0.4203	-0.4200	-0.4198	-0.4195	-0.4192	-0.2430	-0.3367	-0.2429	NN6
3L/4	-0.4186	-0.4190	-0.4195	-0.4201	-0.4205	-0.2430	-0.3368	-0.2431	L
4L/4	-0.4170	-0.4180	-0.4192	-0.4205	-0.4220	-0.2429	-0.3368	-0.2432	
0L/4	-0.4241	-0.4221	-0.4202	-0.4185	-0.4169	-0.2429	-0.3364	-0.2425	
1L/4	-0.4221	-0.4211	-0.4200	-0.4189	-0.4180	-0.2430	-0.3366	-0.2428	U
2L/4	-0.4202	-0.4200	-0.4198	-0.4194	-0.4191	-0.2431	-0.3368	-0.2430	NB6
3L/4	-0.4177	-0.4184	-0.4193	-0.4200	-0.4204	-0.2430	-0.3369	-0.2431	L
4L/4	-0.4169	-0.4180	-0.4191	-0.4204	-0.4219	-0.2430	-0.3369	-0.2432	
0L/4	-0.0004	-0.0002	-0.0001	-0.0001	0.0000	-0.0002	-0.0001	0.0000	
1L/4	-0.0002	-0.0004	-0.0003	-0.0002	-0.0001	-0.0005	-0.0005	-0.0003	U
2L/4	-0.0001	-0.0003	-0.0005	-0.0003	-0.0002	-0.0005	-0.0007	-0.0005	NA6
3L/4	-0.0001	-0.0002	-0.0003	-0.0004	-0.0002	-0.0003	-0.0005	-0.0005	L
4L/4	0.0000	-0.0001	-0.0002	-0.0002	-0.0004	-0.0001	-0.0002	-0.0002	
0L/4	-0.0008	-0.0005	-0.0004	-0.0002	0.0000	-0.0004	-0.0003	-0.0001	
1L/4	-0.0005	-0.0007	-0.0006	-0.0004	-0.0002	-0.0007	-0.0007	-0.0005	U
2L/4	-0.0004	-0.0006	-0.0007	-0.0006	-0.0004	-0.0007	-0.0009	-0.0007	NP6
3L/4	-0.0002	-0.0004	-0.0006	-0.0007	-0.0005	-0.0006	-0.0008	-0.0008	L
4L/4	0.0000	-0.0002	-0.0004	-0.0005	-0.0008	-0.0003	-0.0005	-0.0006	

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TABLE A5 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.4209	-0.4201	-0.4196			-0.2430	-0.3366		
1L/4	-0.4201	-0.4199	-0.4196			-0.2430	-0.3367		U
2L/4	-0.4196	-0.4196	-0.4197			-0.2430	-0.3368		SN6
3L/4	-0.4192	-0.4194	-0.4196			-0.2430	-0.3367		L
4L/4	-0.4189	-0.4192	-0.4196			-0.2429	-0.3366		
0L/4	-0.0004	-0.0002	-0.0002			-0.0002	-0.0002		
1L/4	-0.0002	-0.0004	-0.0003			-0.0005	-0.0005		U
2L/4	-0.0002	-0.0003	-0.0005			-0.0005	-0.0007		SA6
3L/4	-0.0001	-0.0002	-0.0003			-0.0003	-0.0005		L
4L/4	0.0000	-0.0001	-0.0002			-0.0001	-0.0002		
0L/4	-0.0004	-0.0002	-0.0002			-0.0001	-0.0001		
1L/4	-0.0002	-0.0004	-0.0003			-0.0003	-0.0003		U
2L/4	-0.0002	-0.0003	-0.0005			-0.0003	-0.0004		SA6*
3L/4	-0.0001	-0.0002	-0.0003			-0.0002	-0.0003		L
4L/4	-0.0000	-0.0001	-0.0002			-0.0001	-0.0001		
0L/4	-0.2331	-0.2326	-0.2322			-0.1731	-0.2353		
1L/4	-0.2326	-0.2325	-0.2323			-0.1731	-0.2354		U
2L/4	-0.2322	-0.2323	-0.2324			-0.1731	-0.2355		SN4
3L/4	-0.2319	-0.2321	-0.2323			-0.1730	-0.2354		L
4L/4	-0.2317	-0.2319	-0.2322			-0.1729	-0.2353		
0L/4	-0.0003	-0.0002	-0.0001			-0.0001	-0.0001		
1L/4	-0.0002	-0.0003	-0.0002			-0.0003	-0.0003		U
2L/4	-0.0001	-0.0002	-0.0003			-0.0003	-0.0004		SA4
3L/4	-0.0001	-0.0002	-0.0002			-0.0002	-0.0003		L
4L/4	-0.0000	-0.0001	-0.0001			-0.0001	-0.0001		

TABLE A5 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT.	LOADS ARE APPLIED ON THE NODES OF										BRIDGE TYPE
	TOP FLANGE AT X=					BOTTOM FLANGE AT X=					
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4			
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000			
1L/4	-0.2429	-0.2430	-0.2430	-0.2430	-0.2429	-0.8512	-0.9049	-0.5155	U		
2L/4	-0.3363	-0.3365	-0.3367	-0.3368	-0.3368	-0.9049	-1.3556	-0.9050	NN6		
3L/4	-0.2425	-0.2427	-0.2429	-0.2431	-0.2432	-0.5155	-0.9050	-0.8513	L		
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000			
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000			
1L/4	-0.2429	-0.2430	-0.2431	-0.2430	-0.2430	-0.8507	-0.9042	-0.5151	U		
2L/4	-0.3364	-0.3366	-0.3368	-0.3369	-0.3369	-0.9042	-1.3546	-0.9042	NB6		
3L/4	-0.1336	-0.1338	-0.1339	-0.2431	-0.2432	-0.2642	-0.4771	-0.8508	L		
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000			
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000			
1L/4	-0.0002	-0.0005	-0.0005	-0.0003	-0.0001	-0.0018	-0.0010	-0.0007	U		
2L/4	-0.0001	-0.0005	-0.0007	-0.0005	-0.0002	-0.0010	-0.0025	-0.0012	NA6		
3L/4	0.0000	-0.0003	-0.0005	-0.0005	-0.0002	-0.0007	-0.0012	-0.0020	L		
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000			
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000			
1L/4	-0.0004	-0.0007	-0.0007	-0.0006	-0.0003	-0.0045	-0.0015	-0.0009	U		
2L/4	-0.0003	-0.0007	-0.0009	-0.0008	-0.0005	-0.0015	-0.0034	-0.0018	NP6		
3L/4	-0.0001	-0.0005	-0.0007	-0.0008	-0.0006	-0.0009	-0.0018	-0.0050	L		
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000			

TABLE A5 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF								BRIDGE TYPE
	TOP FLANGE AT X=				BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.2430	-0.2430	-0.2430			-0.8511	-0.9048		U
2L/4	-0.3366	-0.3367	-0.3368			-0.9048	-1.3553		SN6
3L/4	-0.2429	-0.2430	-0.2430			-0.5154	-0.9048		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0002	-0.0005	-0.0005			-0.0017	-0.0009		U
2L/4	-0.0002	-0.0005	-0.0007			-0.0009	-0.0023		SA6
3L/4	-0.0001	-0.0003	-0.0005			-0.0006	-0.0009		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0001	-0.0003	-0.0003			-0.7106	-0.7100		U
2L/4	-0.0001	-0.0003	-0.0004			-0.7100	-1.0855		SA6*
3L/4	-0.0001	-0.0002	-0.0003			-0.3749	-0.7100		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.1731	-0.1731	-0.1731			-0.5504	-0.5150		U
2L/4	-0.2353	-0.2354	-0.2355			-0.5150	-0.7963		SN4
3L/4	-0.1729	-0.1730	-0.1731			-0.2652	-0.5150		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0001	-0.0003	-0.0003			-0.0013	-0.0006		U
2L/4	-0.0001	-0.0003	-0.0004			-0.0006	-0.0016		SA4
3L/4	-0.0001	-0.0002	-0.0003			-0.0004	-0.0006		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A6 TRANSVERSE DEFLECTIONS V FOR GIRDER B IN MILIINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.4242	-0.4222	-0.4203	-0.4186	-0.4170	-0.2429	-0.3363	-0.2425	
1L/4	-0.4222	-0.4211	-0.4200	-0.4190	-0.4180	-0.2430	-0.3365	-0.2427	U
2L/4	-0.4203	-0.4200	-0.4198	-0.4195	-0.4192	-0.2430	-0.3367	-0.2429	NN6
3L/4	-0.4186	-0.4190	-0.4195	-0.4200	-0.4205	-0.2430	-0.3368	-0.2431	L
4L/4	-0.4170	-0.4180	-0.4192	-0.4205	-0.4219	-0.2429	-0.3368	-0.2432	
0L/4	-0.4241	-0.4221	-0.4202	-0.4185	-0.4169	-0.2429	-0.3364	-0.2425	
1L/4	-0.4221	-0.4210	-0.4200	-0.4189	-0.4180	-0.2430	-0.3366	-0.2428	U
2L/4	-0.4202	-0.4200	-0.4197	-0.4194	-0.4191	-0.2430	-0.3368	-0.2430	NB6
3L/4	-0.4177	-0.4184	-0.4193	-0.4200	-0.4204	-0.2430	-0.3369	-0.2431	L
4L/4	-0.4169	-0.4180	-0.4191	-0.4204	-0.4218	-0.2430	-0.3369	-0.2432	
0L/4	-0.0003	-0.0002	-0.0001	-0.0001	0.0000	-0.0002	-0.0001	0.0000	
1L/4	-0.0002	-0.0004	-0.0003	-0.0002	-0.0001	-0.0005	-0.0005	-0.0003	U
2L/4	-0.0001	-0.0003	-0.0005	-0.0003	-0.0002	-0.0005	-0.0007	-0.0005	NA6
3L/4	-0.0001	-0.0002	-0.0003	-0.0004	-0.0002	-0.0003	-0.0005	-0.0005	L
4L/4	0.0000	-0.0001	-0.0002	-0.0002	-0.0003	-0.0001	-0.0002	-0.0002	
0L/4	-0.0007	-0.0005	-0.0004	-0.0002	0.0000	-0.0004	-0.0003	-0.0001	
1L/4	-0.0005	-0.0007	-0.0006	-0.0004	-0.0002	-0.0007	-0.0007	-0.0005	U
2L/4	-0.0004	-0.0006	-0.0007	-0.0006	-0.0004	-0.0007	-0.0009	-0.0007	NP6
3L/4	-0.0002	-0.0004	-0.0006	-0.0007	-0.0006	-0.0006	-0.0008	-0.0008	L
4L/4	0.0000	-0.0002	-0.0004	-0.0005	-0.0007	-0.0003	-0.0005	-0.0006	

TABLE A6 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=				BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	
0L/4	-0.4207	-0.4201	-0.4196			-0.2430	-0.3366	
1L/4	-0.4201	-0.4199	-0.4196			-0.2430	-0.3367	U
2L/4	-0.4196	-0.4196	-0.4197			-0.2430	-0.3368	SN6
3L/4	-0.4192	-0.4194	-0.4196			-0.2430	-0.3367	L
4L/4	-0.4189	-0.4192	-0.4196			-0.2429	-0.3366	
0L/4	-0.0003	-0.0002	-0.0001			-0.0002	-0.0001	
1L/4	-0.0002	-0.0004	-0.0003			-0.0005	-0.0005	U
2L/4	-0.0002	-0.0003	-0.0005			-0.0005	-0.0007	SA6
3L/4	-0.0001	-0.0002	-0.0003			-0.0003	-0.0005	L
4L/4	0.0000	-0.0001	-0.0001			-0.0001	-0.0001	
0L/4	-0.0003	-0.0002	-0.0001			-0.0001	-0.0001	
1L/4	-0.0002	-0.0004	-0.0003			-0.0003	-0.0003	U
2L/4	-0.0002	-0.0003	-0.0005			-0.0003	-0.0004	SA6*
3L/4	-0.0001	-0.0002	-0.0003			-0.0002	-0.0003	L
4L/4	-0.0000	-0.0001	-0.0001			-0.0001	-0.0001	
0L/4	-0.2330	-0.2326	-0.2322			-0.1731	-0.2353	
1L/4	-0.2326	-0.2325	-0.2323			-0.1731	-0.2354	U
2L/4	-0.2322	-0.2323	-0.2323			-0.1731	-0.2355	SN4
3L/4	-0.2319	-0.2321	-0.2323			-0.1730	-0.2354	L
4L/4	-0.2317	-0.2319	-0.2322			-0.1729	-0.2353	
0L/4	-0.0002	-0.0002	-0.0001			-0.0001	-0.0001	
1L/4	-0.0002	-0.0003	-0.0002			-0.0003	-0.0003	U
2L/4	-0.0001	-0.0002	-0.0003			-0.0003	-0.0004	SA4
3L/4	-0.0001	-0.0002	-0.0002			-0.0002	-0.0003	L
4L/4	-0.0000	-0.0001	-0.0001			-0.0001	-0.0001	

TABLE A6 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.2431	-0.2431	-0.2432	-0.2431	-0.2431	-0.1409	-0.1952	-0.1409	U
2L/4	-0.3365	-0.3368	-0.3369	-0.3370	-0.3371	-0.1952	-0.2706	-0.1953	NN6
3L/4	-0.2426	-0.2429	-0.2431	-0.2433	-0.2434	-0.1409	-0.1953	-0.1409	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.2430	-0.2431	-0.2431	-0.2431	-0.2430	-0.1409	-0.1952	-0.1409	U
2L/4	-0.3364	-0.3367	-0.3369	-0.3369	-0.3370	-0.1952	-0.2706	-0.1952	NB6
3L/4	-0.1336	-0.1338	-0.1339	-0.2432	-0.2433	-0.0776	-0.1076	-0.1409	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0002	-0.0005	-0.0005	-0.0003	-0.0001	-0.0012	-0.0010	-0.0007	U
2L/4	-0.0001	-0.0005	-0.0007	-0.0005	-0.0002	-0.0010	-0.0019	-0.0011	NA6
3L/4	0.0000	-0.0003	-0.0003	-0.0005	-0.0002	-0.0007	-0.0011	-0.0015	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0004	-0.0007	-0.0007	-0.0006	-0.0003	-0.0018	-0.0013	-0.0009	U
2L/4	-0.0003	-0.0007	-0.0009	-0.0008	-0.0005	-0.0013	-0.0026	-0.0016	NP6
3L/4	-0.0001	-0.0005	-0.0007	-0.0008	-0.0006	-0.0009	-0.0016	-0.0021	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	

TABLE A6 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SN6 L
1L/4	-0.2431	-0.2432	-0.2432			-0.1408	-0.1952		
2L/4	-0.3369	-0.3370	-0.3370			-0.1952	-0.2705		
3L/4	-0.2430	-0.2431	-0.2432			-0.1408	-0.1952		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SA6 L
1L/4	-0.0002	-0.0005	-0.0005			-0.0012	-0.0009		
2L/4	-0.0002	-0.0005	-0.0007			-0.0009	-0.0017		
3L/4	-0.0001	-0.0003	-0.0005			-0.0006	-0.0009		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SA6* L
1L/4	-0.0001	-0.0003	-0.0003			-0.0002	-0.0003		
2L/4	-0.0001	-0.0003	-0.0004			-0.0003	-0.0004		
3L/4	-0.0001	-0.0002	-0.0003			-0.0002	-0.0003		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SN4 L
1L/4	-0.1732	-0.1732	-0.1732			-0.1290	-0.1755		
2L/4	-0.2355	-0.2356	-0.2356			-0.1755	-0.2388		
3L/4	-0.1730	-0.1731	-0.1732			-0.1291	-0.1755		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SA4 L
1L/4	-0.0001	-0.0003	-0.0003			-0.0008	-0.0006		
2L/4	-0.0001	-0.0003	-0.0004			-0.0006	-0.0011		
3L/4	-0.0001	-0.0002	-0.0003			-0.0004	-0.0006		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A7 TRANSVERSE DEFLECTIONS V FOR GIRDER C IN MILIINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.4241	-0.4222	-0.4203	-0.4186	-0.4170	-0.2429	-0.3363	-0.2425	
1L/4	-0.4222	-0.4211	-0.4200	-0.4190	-0.4180	-0.2430	-0.3365	-0.2427	U
2L/4	-0.4203	-0.4200	-0.4198	-0.4195	-0.4192	-0.2430	-0.3367	-0.2429	NN6
3L/4	-0.4186	-0.4190	-0.4195	-0.4200	-0.4205	-0.2430	-0.3368	-0.2431	L
4L/4	-0.4170	-0.4180	-0.4192	-0.4205	-0.4218	-0.2429	-0.3368	-0.2432	
0L/4	-0.4240	-0.4221	-0.4202	-0.4185	-0.4169	-0.2430	-0.3364	-0.2425	
1L/4	-0.4221	-0.4210	-0.4200	-0.4189	-0.4180	-0.2430	-0.3366	-0.2428	U
2L/4	-0.4202	-0.4200	-0.4197	-0.4194	-0.4191	-0.2430	-0.3368	-0.2430	NB6
3L/4	-0.4177	-0.4184	-0.4193	-0.4199	-0.4204	-0.2430	-0.3369	-0.2431	L
4L/4	-0.4169	-0.4180	-0.4191	-0.4204	-0.4218	-0.2430	-0.3369	-0.2432	
0L/4	-0.0003	-0.0002	-0.0001	-0.0001	0.0000	-0.0002	-0.0001	0.0000	
1L/4	-0.0002	-0.0004	-0.0003	-0.0002	-0.0001	-0.0005	-0.0005	-0.0003	U
2L/4	-0.0001	-0.0003	-0.0004	-0.0003	-0.0002	-0.0005	-0.0007	-0.0005	NA6
3L/4	-0.0001	-0.0002	-0.0003	-0.0004	-0.0002	-0.0003	-0.0005	-0.0005	L
4L/4	0.0000	-0.0001	-0.0002	-0.0002	-0.0003	-0.0001	-0.0002	-0.0002	
0L/4	-0.0007	-0.0005	-0.0004	-0.0002	0.0000	-0.0004	-0.0003	-0.0001	
1L/4	-0.0006	-0.0006	-0.0006	-0.0004	-0.0002	-0.0007	-0.0007	-0.0005	U
2L/4	-0.0004	-0.0006	-0.0007	-0.0006	-0.0004	-0.0007	-0.0009	-0.0007	NP6
3L/4	-0.0002	-0.0004	-0.0006	-0.0006	-0.0006	-0.0006	-0.0008	-0.0008	L
4L/4	0.0000	-0.0002	-0.0004	-0.0006	-0.0007	-0.0003	-0.0005	-0.0006	

TABLE A7 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.4207	-0.4201	-0.4196			-0.2430	-0.3367		U SN6 L
1L/4	-0.4201	-0.4199	-0.4196			-0.2430	-0.3367		
2L/4	-0.4196	-0.4196	-0.4197			-0.2430	-0.3368		
3L/4	-0.4192	-0.4194	-0.4196			-0.2430	-0.3367		
4L/4	-0.4189	-0.4192	-0.4196			-0.2429	-0.3367		
0L/4	-0.0002	-0.0002	-0.0001			-0.0002	-0.0001		U SA6 L
1L/4	-0.0002	-0.0004	-0.0003			-0.0005	-0.0005		
2L/4	-0.0002	-0.0003	-0.0004			-0.0005	-0.0007		
3L/4	-0.0001	-0.0002	-0.0003			-0.0003	-0.0005		
4L/4	0.0000	-0.0001	-0.0001			-0.0001	-0.0001		
0L/4	-0.0002	-0.0002	-0.0001			-0.0001	-0.0001		U SA6* L
1L/4	-0.0002	-0.0004	-0.0003			-0.0002	-0.0003		
2L/4	-0.0002	-0.0003	-0.0004			-0.0003	-0.0004		
3L/4	-0.0001	-0.0002	-0.0003			-0.0002	-0.0003		
4L/4	-0.0000	-0.0001	-0.0001			-0.0001	-0.0001		
0L/4	-0.2329	-0.2326	-0.2322			-0.1731	-0.2353		U SN4 L
1L/4	-0.2326	-0.2324	-0.2323			-0.1731	-0.2354		
2L/4	-0.2322	-0.2323	-0.2323			-0.1731	-0.2355		
3L/4	-0.2319	-0.2321	-0.2323			-0.1730	-0.2354		
4L/4	-0.2317	-0.2319	-0.2322			-0.1729	-0.2353		
0L/4	-0.0002	-0.0002	-0.0001			-0.0001	-0.0001		U SA4 L
1L/4	-0.0002	-0.0002	-0.0002			-0.0003	-0.0003		
2L/4	-0.0001	-0.0002	-0.0003			-0.0003	-0.0004		
3L/4	-0.0001	-0.0002	-0.0002			-0.0002	-0.0003		
4L/4	-0.0000	-0.0001	-0.0001			-0.0001	-0.0001		

TABLE A7 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.2431	-0.2431	-0.2432	-0.2431	-0.2431	-0.1407	-0.1950	-0.1407	U
2L/4	-0.3365	-0.3368	-0.3369	-0.3370	-0.3371	-0.1950	-0.2702	-0.1950	NN6
3L/4	-0.2427	-0.2429	-0.2431	-0.2433	-0.2434	-0.1407	-0.1950	-0.1407	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.2430	-0.2431	-0.2431	-0.2431	-0.2430	-0.1409	-0.1952	-0.1409	U
2L/4	-0.3364	-0.3367	-0.3368	-0.3369	-0.3370	-0.1952	-0.2706	-0.1952	NB6
3L/4	-0.1336	-0.1338	-0.1339	-0.2432	-0.2433	-0.0776	-0.1076	-0.1409	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0002	-0.0005	-0.0005	-0.0003	-0.0001	-0.0010	-0.0010	-0.0007	U
2L/4	-0.0001	-0.0005	-0.0007	-0.0005	-0.0002	-0.0010	-0.0016	-0.0011	NA6
3L/4	0.0000	-0.0003	-0.0005	-0.0005	-0.0002	-0.0007	-0.0011	-0.0012	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0004	-0.0007	-0.0007	-0.0006	-0.0003	-0.0011	-0.0011	-0.0008	U
2L/4	-0.0003	-0.0007	-0.0009	-0.0008	-0.0005	-0.0011	-0.0017	-0.0013	NP6
3L/4	-0.0001	-0.0005	-0.0007	-0.0008	-0.0006	-0.0008	-0.0013	-0.0013	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	

TABLE A7 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.2431	-0.2432	-0.2432			-0.1407	-0.1950		U
2L/4	-0.3369	-0.3370	-0.3370			-0.1950	-0.2703		SN6
3L/4	-0.2430	-0.2431	-0.2432			-0.1407	-0.1950		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0002	-0.0005	-0.0005			-0.0009	-0.0009		U
2L/4	-0.0002	-0.0005	-0.0007			-0.0009	-0.0014		SA6
3L/4	-0.0001	-0.0003	-0.0005			-0.0006	-0.0009		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0001	-0.0002	-0.0003			-0.0001	-0.0002		U
2L/4	-0.0001	-0.0003	-0.0004			-0.0002	-0.0003		SA6*
3L/4	-0.0001	-0.0002	-0.0003			-0.0001	-0.0002		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.1732	-0.1732	-0.1732			-0.1290	-0.1754		U
2L/4	-0.2355	-0.2356	-0.2356			-0.1754	-0.2386		SN4
3L/4	-0.1731	-0.1731	-0.1732			-0.1290	-0.1754		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0001	-0.0003	-0.0003			-0.0006	-0.0006		U
2L/4	-0.0001	-0.0003	-0.0004			-0.0006	-0.0009		SA4
3L/4	-0.0001	-0.0002	-0.0003			-0.0004	-0.0006		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A8 TRANSVERSE DEFLECTIONS V FOR GIRDER D IN MILIINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.4241	-0.4222	-0.4203	-0.4186	-0.4170	-0.2429	-0.3363	-0.2425	
1L/4	-0.4222	-0.4211	-0.4200	-0.4190	-0.4180	-0.2430	-0.3365	-0.2427	U
2L/4	-0.4203	-0.4200	-0.4198	-0.4195	-0.4192	-0.2430	-0.3367	-0.2429	NN6
3L/4	-0.4186	-0.4190	-0.4195	-0.4200	-0.4205	-0.2430	-0.3368	-0.2431	L
4L/4	-0.4170	-0.4180	-0.4192	-0.4205	-0.4218	-0.2429	-0.3368	-0.2432	
0L/4	-0.4240	-0.4221	-0.4202	-0.4185	-0.4169	-0.2430	-0.3364	-0.2425	
1L/4	-0.4221	-0.4210	-0.4200	-0.4189	-0.4180	-0.2430	-0.3366	-0.2428	U
2L/4	-0.4202	-0.4200	-0.4197	-0.4194	-0.4191	-0.2430	-0.3368	-0.2430	NB6
3L/4	-0.4177	-0.4184	-0.4193	-0.4199	-0.4204	-0.2430	-0.3369	-0.2431	L
4L/4	-0.4169	-0.4180	-0.4191	-0.4204	-0.4218	-0.2430	-0.3369	-0.2432	
0L/4	-0.0003	-0.0002	-0.0001	-0.0001	0.0000	-0.0002	-0.0001	0.0000	
1L/4	-0.0002	-0.0004	-0.0003	-0.0002	-0.0001	-0.0005	-0.0005	-0.0003	U
2L/4	-0.0001	-0.0003	-0.0004	-0.0003	-0.0002	-0.0005	-0.0007	-0.0005	NA6
3L/4	-0.0001	-0.0002	-0.0003	-0.0004	-0.0002	-0.0003	-0.0005	-0.0005	L
4L/4	0.0000	-0.0001	-0.0002	-0.0002	-0.0003	-0.0001	-0.0002	-0.0002	
0L/4	-0.0007	-0.0006	-0.0004	-0.0002	0.0000	-0.0004	-0.0003	-0.0001	
1L/4	-0.0006	-0.0006	-0.0006	-0.0004	-0.0002	-0.0007	-0.0007	-0.0005	U
2L/4	-0.0004	-0.0006	-0.0007	-0.0006	-0.0004	-0.0007	-0.0009	-0.0007	NP6
3L/4	-0.0002	-0.0004	-0.0006	-0.0006	-0.0006	-0.0006	-0.0008	-0.0008	L
4L/4	0.0000	-0.0002	-0.0004	-0.0006	-0.0007	-0.0003	-0.0005	-0.0006	

TABLE A8 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF								BRIDGE TYPE
	TOP FLANGE AT X=				BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.4207	-0.4201	-0.4196			-0.2430	-0.3367		
1L/4	-0.4201	-0.4199	-0.4196			-0.2430	-0.3367		U
2L/4	-0.4196	-0.4196	-0.4197			-0.2430	-0.3368		SN6
3L/4	-0.4192	-0.4194	-0.4196			-0.2430	-0.3367		L
4L/4	-0.4189	-0.4192	-0.4196			-0.2429	-0.3367		
0L/4	-0.0002	-0.0002	-0.0002			-0.0002	-0.0002		
1L/4	-0.0002	-0.0004	-0.0003			-0.0005	-0.0005		U
2L/4	-0.0002	-0.0003	-0.0004			-0.0005	-0.0007		SA6
3L/4	-0.0001	-0.0002	-0.0003			-0.0003	-0.0005		L
4L/4	0.0000	-0.0001	-0.0002			-0.0001	-0.0002		
0L/4	-0.0002	-0.0002	-0.0002			-0.0001	-0.0001		
1L/4	-0.0002	-0.0004	-0.0003			-0.0002	-0.0003		U
2L/4	-0.0002	-0.0003	-0.0004			-0.0003	-0.0004		SA6*
3L/4	-0.0001	-0.0002	-0.0003			-0.0002	-0.0003		L
4L/4	-0.0000	-0.0001	-0.0002			-0.0001	-0.0001		
0L/4	-0.2329	-0.2326	-0.2322			-0.1731	-0.2353		
1L/4	-0.2326	-0.2324	-0.2323			-0.1731	-0.2354		U
2L/4	-0.2322	-0.2323	-0.2323			-0.1731	-0.2355		SN4
3L/4	-0.2319	-0.2321	-0.2323			-0.1730	-0.2354		L
4L/4	-0.2317	-0.2319	-0.2322			-0.1729	-0.2353		
0L/4	-0.0002	-0.0002	-0.0001			-0.0001	-0.0001		
1L/4	-0.0002	-0.0002	-0.0002			-0.0003	-0.0003		U
2L/4	-0.0001	-0.0002	-0.0003			-0.0003	-0.0004		SA4
3L/4	-0.0001	-0.0002	-0.0002			-0.0002	-0.0003		L
4L/4	-0.0000	-0.0001	-0.0001			-0.0001	-0.0001		

TABLE A8 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					NODES OF BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.2429	-0.2430	-0.2430	-0.2430	-0.2429	-0.1406	-0.1949	-0.1406	U
2L/4	-0.3363	-0.3365	-0.3367	-0.3368	-0.3368	-0.1949	-0.2700	-0.1949	NN6
3L/4	-0.2425	-0.2427	-0.2429	-0.2431	-0.2432	-0.1406	-0.1949	-0.1406	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.2430	-0.2430	-0.2430	-0.2430	-0.2430	-0.1408	-0.1951	-0.1408	U
2L/4	-0.3364	-0.3366	-0.3368	-0.3368	-0.3369	-0.1951	-0.2704	-0.1951	NB6
3L/4	-0.1336	-0.1337	-0.1339	-0.2431	-0.2432	-0.0776	-0.1075	-0.1408	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0002	-0.0005	-0.0005	-0.0003	-0.0001	-0.0009	-0.0010	-0.0006	U
2L/4	-0.0001	-0.0005	-0.0007	-0.0005	-0.0002	-0.0010	-0.0016	-0.0011	NA6
3L/4	0.0000	-0.0003	-0.0005	-0.0005	-0.0002	-0.0006	-0.0011	-0.0011	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0004	-0.0007	-0.0007	-0.0006	-0.0003	-0.0009	-0.0010	-0.0008	U
2L/4	-0.0003	-0.0007	-0.0009	-0.0008	-0.0005	-0.0010	-0.0015	-0.0012	NP6
3L/4	-0.0001	-0.0005	-0.0007	-0.0008	-0.0006	-0.0008	-0.0012	-0.0011	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	

TABLE A8 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.2430	-0.2430	-0.2430			-0.1407	-0.1950		U
2L/4	-0.3367	-0.3367	-0.3368			-0.1950	-0.2702		SN6
3L/4	-0.2429	-0.2430	-0.2430			-0.1407	-0.1950		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0002	-0.0005	-0.0005			-0.0009	-0.0009		U
2L/4	-0.0002	-0.0005	-0.0007			-0.0009	-0.0014		SA6
3L/4	-0.0001	-0.0003	-0.0005			-0.0006	-0.0009		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0001	-0.0002	-0.0003			-0.0002	-0.0002		U
2L/4	-0.0001	-0.0003	-0.0004			-0.0002	-0.0003		SA6*
3L/4	-0.0001	-0.0002	-0.0003			-0.0002	-0.0002		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.1731	-0.1731	-0.1731			-0.1289	-0.1754		U
2L/4	-0.2353	-0.2354	-0.2355			-0.1754	-0.2386		SN4
3L/4	-0.1729	-0.1730	-0.1731			-0.1289	-0.1754		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0001	-0.0003	-0.0003			-0.0005	-0.0005		U
2L/4	-0.0001	-0.0003	-0.0004			-0.0005	-0.0008		SA4
3L/4	-0.0001	-0.0002	-0.0003			-0.0004	-0.0005		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A9 VERTICAL DEFLECTIONS W FOR GIRDER A IN MICROINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0767	0.0528	0.0271	0.0015	-0.0241	-0.0079	-0.0086	-0.0055	
1L/4	2.7744	1.5667	0.5652	-0.2718	-1.0265	-0.4929	-0.8220	-0.6549	U
2L/4	3.3240	1.9474	0.7445	-0.1867	-0.9504	-0.7844	-1.3408	-1.0755	NN6
3L/4	2.2051	1.3449	0.5707	-0.0393	-0.4497	-0.5384	-0.9698	-0.8198	L
4L/4	0.0495	0.0383	0.0271	0.0161	0.0030	-0.0056	-0.0159	-0.0182	
0L/4	0.0601	0.0511	0.0257	0.0002	-0.0253	-0.0003	-0.0005	-0.0012	
1L/4	2.7406	1.5363	0.5415	-0.2920	-1.0466	-0.2780	-0.5274	-0.4531	U
2L/4	3.2878	1.9167	0.7046	-0.2177	-0.9809	-0.4162	-0.8046	-0.6882	NB6
3L/4	1.1847	0.7324	0.3193	-0.0758	-0.4829	-0.1332	-0.2916	-0.5014	L
4L/4	0.0479	0.0367	0.0256	0.0144	-0.0133	0.0027	-0.0018	-0.0059	
0L/4	0.0539	0.0319	0.0205	0.0095	-0.0014	0.0012	0.0098	0.0059	
1L/4	0.1068	-0.1691	-0.2759	-0.2230	-0.0878	-0.9458	-1.0804	-0.7028	U
2L/4	0.1046	-0.2256	-0.4033	-0.3313	-0.0942	-1.0328	-1.9057	-1.3299	NA6
3L/4	0.0631	-0.1447	-0.2782	-0.2497	-0.0437	-0.6262	-1.2769	-1.3267	L
4L/4	0.0017	0.0113	0.0206	0.0302	0.0507	0.0140	0.0146	0.0001	
0L/4	0.0460	0.0311	0.0195	0.0081	-0.0032	-0.0107	0.0071	0.0055	
1L/4	0.2342	-0.1110	-0.2755	-0.2826	-0.2089	-1.1127	-1.2706	-0.8489	U
2L/4	0.2468	-0.1562	-0.4056	-0.3991	-0.2295	-1.1591	-2.2684	-1.6222	NP6
3L/4	0.1513	-0.1000	-0.2770	-0.2946	-0.1256	-0.6717	-1.5009	-1.6567	L
4L/4	0.0042	0.0120	0.0196	0.0273	0.0384	0.0139	0.0089	-0.0172	

TABLE A9 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF								BRIDGE TYPE
	TOP FLANGE AT X=				BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0298	0.0248	0.0175			-0.0073	-0.0044		
1L/4	0.7224	0.3366	0.1457			-0.4170	-0.5599		U
2L/4	0.6032	0.3160	0.1867			-0.6301	-0.9105		SN6
3L/4	0.1462	0.1092	0.1457			-0.3674	-0.5599		L
4L/4	0.0027	0.0103	0.0175			-0.0003	-0.0044		
0L/4	0.0516	0.0308	0.0206			0.0042	0.0181		
1L/4	0.0548	-0.1942	-0.2732			-0.8797	-0.8954		U
2L/4	0.0014	-0.2754	-0.3980			-0.8996	-1.5328		SA6
3L/4	-0.0406	-0.1949	-0.2732			-0.4906	-0.8954		L
4L/4	0.0007	0.0108	0.0206			0.0153	0.0181		
0L/4	0.0521	0.0302	0.0207			-0.0053	-0.0022		
1L/4	0.0640	-0.2027	-0.2760			-0.6724	-0.8981		U
2L/4	0.0089	-0.2780	-0.4072			-0.9738	-1.3868		SA6*
3L/4	-0.0383	-0.1951	-0.2760			-0.6001	-0.8981		L
4L/4	0.0007	0.0109	0.0207			0.0010	-0.0022		
0L/4	0.0254	0.0200	0.0127			-0.0039	0.0016		
1L/4	0.5866	0.2271	0.0400			-0.5704	-0.7552		U
2L/4	0.3762	0.1312	0.0243			-0.8545	-1.2426		SN4
3L/4	-0.0145	-0.0185	0.0400			-0.4863	-0.7552		L
4L/4	-0.0018	0.0056	0.0127			0.0041	0.0016		
0L/4	0.0402	0.0214	0.0139			0.0040	0.0146		
1L/4	0.0565	-0.1531	-0.2260			-0.8089	-0.8158		U
2L/4	-0.0005	-0.2273	-0.3248			-0.8188	-1.4197		SA4
3L/4	-0.0449	-0.1677	-0.2260			-0.4360	-0.8158		L
4L/4	-0.0002	0.0069	0.0139			0.0113	0.0146		

TABLE A9 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	2.7586	1.5560	0.5621	-0.2672	-1.0141	-0.4921	-0.8207	-0.6537	U
2L/4	3.3131	1.9397	0.7403	-0.1850	-0.9427	-0.7810	-1.3348	-1.0709	NN6
3L/4	2.1986	1.3400	0.5675	-0.0406	-0.4467	-0.5367	-0.9659	-0.8157	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	2.7251	1.5264	0.5385	-0.2871	-1.0339	-0.2796	-0.5288	-0.4534	U
2L/4	3.2771	1.9091	0.7011	-0.2160	-0.9730	-0.4179	-0.8060	-0.6887	NB6
3L/4	1.1801	0.7287	0.3165	-0.0762	-0.4795	-0.1333	-0.2911	-0.5020	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	0.1065	-0.1719	-0.2772	-0.2234	-0.0871	-0.9579	-1.0838	-0.7048	U
2L/4	0.1043	-0.2271	-0.4063	-0.3324	-0.0937	-1.0342	-1.9153	-1.3302	NA6
3L/4	0.0629	-0.1455	-0.2795	-0.2520	-0.0431	-0.6269	-1.2769	-1.3347	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	0.2330	-0.1137	-0.2769	-0.2825	-0.2073	-1.1437	-1.2748	-0.8509	U
2L/4	0.2459	-0.1580	-0.4083	-0.3999	-0.2284	-1.1612	-2.2962	-1.6227	NP6
3L/4	0.1508	-0.1009	-0.2783	-0.2963	-0.1248	-0.6728	-1.5013	-1.6826	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	

TABLE A9 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SN6 L
1L/4	0.7186	0.3331	0.1450			-0.4161	-0.5594		
2L/4	0.6042	0.3154	0.1850			-0.6272	-0.9062		
3L/4	0.1518	0.1116	0.1450			-0.3673	-0.5594		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SA6 L
1L/4	0.0546	-0.1969	-0.2745			-0.8918	-0.8987		
2L/4	0.0016	-0.2767	-0.4010			-0.9016	-1.5440		
3L/4	-0.0399	-0.1953	-0.2745			-0.4926	-0.8987		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SA6* L
1L/4	-0.0636	-0.2049	-0.2774			-0.6717	-0.8982		
2L/4	0.0090	-0.2794	-0.4097			-0.9714	-1.3832		
3L/4	-0.0375	-0.1955	-0.2774			-0.6006	-0.8982		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SN4 L
1L/4	0.5833	0.2242	0.0393			-0.5696	-0.7550		
2L/4	0.3768	0.1308	0.0233			-0.8517	-1.2383		
3L/4	-0.0103	-0.0168	0.0393			-0.4865	-0.7550		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		U SA4 L
1L/4	0.0563	-0.1551	-0.2268			-0.8179	-0.8181		
2L/4	-0.0003	-0.2282	-0.3270			-0.8200	-1.4277		
3L/4	-0.0443	-0.1679	-0.2268			-0.4374	-0.8181		
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A10 VERTICAL DEFLECTIONS W FOR GIRDER B IN MICROINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0085	0.0021	-0.0049	-0.0120	-0.0192	0.0142	0.0106	0.0029	
1L/4	0.8431	0.4780	0.1333	-0.1500	-0.4050	0.3335	0.3699	0.1989	U
2L/4	1.0307	0.5853	0.1893	-0.1328	-0.3992	0.5262	0.7007	0.4611	NN6
3L/4	0.6584	0.3753	0.1173	-0.0786	-0.2464	0.3659	0.5402	0.4092	L
4L/4	0.0005	-0.0023	-0.0051	-0.0080	-0.0114	0.0092	0.0182	0.0189	
0L/4	0.0402	0.0095	0.0016	-0.0061	-0.0138	0.0038	0.0011	-0.0012	
1L/4	0.8973	0.5295	0.1785	-0.1059	-0.3592	0.0476	-0.0109	-0.0540	U
2L/4	1.0894	0.6414	0.2595	-0.0700	-0.3342	0.0522	0.0102	-0.0348	NB6
3L/4	0.3750	0.2248	0.0901	-0.0071	-0.1763	0.0170	0.0158	0.0009	L
4L/4	0.0079	0.0048	0.0016	-0.0012	0.0186	0.0009	0.0023	0.0035	
0L/4	0.0014	0.0004	0.0007	0.0002	-0.0005	0.0050	-0.0015	-0.0008	
1L/4	0.0246	-0.0569	-0.0926	-0.0737	-0.0279	-0.1184	-0.2964	-0.2266	U
2L/4	0.0349	-0.0759	-0.1363	-0.1121	-0.0298	-0.2806	-0.4037	-0.3685	NA6
3L/4	0.0223	-0.0478	-0.0948	-0.0865	-0.0211	-0.2013	-0.3508	-0.2147	L
4L/4	0.0003	0.0006	0.0007	0.0000	0.0007	-0.0011	-0.0034	0.0025	
0L/4	-0.0004	0.0002	0.0000	-0.0005	-0.0011	0.0154	-0.0017	-0.0031	
1L/4	0.0584	-0.0329	-0.0926	-0.0947	-0.0688	0.1583	-0.1377	-0.1926	U
2L/4	0.0776	-0.0523	-0.1332	-0.1349	-0.0781	-0.0983	-0.0078	-0.1765	NP6
3L/4	0.0503	-0.0342	-0.0953	-0.0977	-0.0559	-0.1333	-0.1366	0.1288	L
4L/4	0.0006	0.0003	0.0000	-0.0007	-0.0019	-0.0023	-0.0021	0.0144	

TABLE A10 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.0042	-0.0055	-0.0076			0.0147	0.0122		U SN6 L
1L/4	0.1570	0.0669	-0.0056			0.3021	0.3697		
2L/4	0.1261	0.0455	0.0110			0.4263	0.6091		
3L/4	-0.0118	-0.0203	-0.0056			0.2267	0.3697		
4L/4	-0.0121	-0.0098	-0.0076			0.0047	0.0122		
0L/4	0.0013	0.0004	0.0007			0.0051	-0.0011		U SA6 L
1L/4	0.0067	-0.0655	-0.0920			-0.0973	-0.2364		
2L/4	-0.0004	-0.0928	-0.1344			-0.2380	-0.2854		
3L/4	-0.0130	-0.0645	-0.0920			-0.1585	-0.2364		
4L/4	-0.0004	0.0002	0.0007			-0.0003	-0.0011		
0L/4	0.0008	0.0009	0.0006			0.0159	0.0155		U SA6* L
1L/4	-0.0046	-0.0562	-0.0892			0.2492	0.3019		
2L/4	-0.0082	-0.0897	-0.1242			0.3477	0.5003		
3L/4	-0.0154	-0.0644	-0.0892			0.1818	0.3019		
4L/4	-0.0004	0.0001	0.0006			0.0079	0.0155		
0L/4	-0.0036	-0.0044	-0.0064			0.0141	0.0074		U SN4 L
1L/4	0.1069	0.0474	-0.0273			0.3974	0.4483		
2L/4	0.0609	0.0068	-0.0164			0.5227	0.7538		
3L/4	-0.0578	-0.0522	-0.0273			0.2467	0.4483		
4L/4	-0.0108	-0.0086	-0.0064			-0.0004	0.0074		
0L/4	0.0005	0.0001	0.0005			0.0054	-0.0005		U SA4 L
1L/4	0.0008	-0.0519	-0.0768			-0.0388	-0.1850		
2L/4	-0.0031	-0.0781	-0.1124			-0.1866	-0.1965		
3L/4	-0.0144	-0.0556	-0.0768			-0.1352	-0.1850		
4L/4	-0.0005	0.0001	0.0005			-0.0002	-0.0005		

TABLE A10 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF									BRIDGE TYPE
	TOP FLANGE AT X=					BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4		
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
1L/4	0.8378	0.4753	0.1326	-0.1481	-0.4005	0.3318	0.3691	0.1992	U	
2L/4	1.0269	0.5827	0.1883	-0.1323	-0.3967	0.5224	0.6953	0.4576	NN6	
3L/4	0.6557	0.3742	0.1168	-0.0780	-0.2453	0.3639	0.5363	0.4055	L	
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
1L/4	0.8907	0.5247	0.1767	-0.1054	-0.3562	0.0502	-0.0071	-0.0514	U	
2L/4	1.0849	0.6383	0.2571	-0.0701	-0.3327	0.0565	0.0166	-0.0300	NB6	
3L/4	0.3746	0.2246	0.0902	-0.0089	-0.1768	0.0166	0.0148	0.0043	L	
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
1L/4	0.0243	-0.0579	-0.0931	-0.0738	-0.0278	-0.1344	-0.2978	-0.2269	U	
2L/4	0.0347	-0.0764	-0.1374	-0.1124	-0.0298	-0.2816	-0.4197	-0.3692	NA6	
3L/4	0.0222	-0.0481	-0.0952	-0.0873	-0.0212	-0.2013	-0.3516	-0.2305	L	
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
1L/4	0.0578	-0.0336	-0.0931	-0.0946	-0.0684	0.1353	-0.1397	-0.1925	U	
2L/4	0.0772	-0.0528	-0.1338	-0.1351	-0.0779	-0.1007	-0.0326	-0.1791	NP6	
3L/4	0.0501	-0.0344	-0.0957	-0.0979	-0.0560	-0.1336	-0.1394	0.1041	L	
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		

TABLE A10 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF								BRIDGE TYPE
	TOP FLANGE AT X=				BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	0.1557	0.0666	-0.0055			0.3000	0.3681		U
2L/4	0.1263	0.0453	0.0110			0.4227	0.6038		SN6
3L/4	-0.0096	-0.0192	-0.0055			0.2263	0.3681		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	0.0064	-0.0666	-0.0924			-0.1134	-0.2378		U
2L/4	-0.0004	-0.0932	-0.1355			-0.2392	-0.3018		SA6
3L/4	-0.0128	-0.0646	-0.0924			-0.1588	-0.2378		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0051	-0.0565	-0.0897			0.2468	0.2998		U
2L/4	-0.0083	-0.0902	-0.1245			0.3440	0.4947		SA6*
3L/4	-0.0152	-0.0646	-0.0897			0.1809	0.2998		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	0.1056	0.0472	-0.0273			0.3951	0.4470		U
2L/4	0.0609	0.0065	-0.0164			0.5195	0.7487		SN4
3L/4	-0.0562	-0.0513	-0.0273			0.2467	0.4470		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	0.0005	-0.0527	-0.0771			-0.0512	-0.1862		U
2L/4	-0.0032	-0.0784	-0.1131			-0.1876	-0.2094		SA4
3L/4	-0.0142	-0.0556	-0.0771			-0.1354	-0.1862		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE All VERTICAL DEFLECTIONS W FOR GIRDER C IN MICROINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF								BRIDGE TYPE
	TOP FLANGE AT X=				BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.0096	-0.0021	0.0049	0.0120	0.0194	0.0057	0.0100	0.0082	
1L/4	-0.8785	-0.4586	-0.1249	0.1535	0.4031	0.2012	0.3451	0.2809	U
2L/4	-1.0516	-0.5766	-0.1680	0.1411	0.3885	0.3464	0.5689	0.4511	NN6
3L/4	-0.6666	-0.3718	-0.1093	0.0974	0.2212	0.2755	0.4473	0.3514	L
4L/4	-0.0003	0.0023	0.0051	0.0081	0.0107	0.0112	0.0167	0.0121	
0L/4	-0.0187	-0.0094	-0.0018	0.0059	0.0138	0.0036	0.0048	0.0036	
1L/4	-0.9089	-0.4943	-0.1633	0.1106	0.3592	0.1330	0.2298	0.1871	U
2L/4	-1.0934	-0.6264	-0.2165	0.0867	0.3323	0.1966	0.3515	0.2892	NB6
3L/4	-0.3750	-0.2248	-0.0825	0.0461	0.1692	0.0732	0.1394	0.2197	L
4L/4	-0.0078	-0.0049	-0.0018	0.0015	0.0030	0.0030	0.0061	0.0060	
0L/4	-0.0013	-0.0011	-0.0006	-0.0001	0.0006	0.0063	0.0000	-0.0010	
1L/4	-0.0367	0.0629	0.0961	0.0766	0.0301	0.3393	0.3785	0.2389	U
2L/4	-0.0349	0.0792	0.1425	0.1143	0.0322	0.3632	0.6726	0.4665	NA6
3L/4	-0.0204	0.0507	0.0971	0.0896	0.0147	0.2142	0.4494	0.4760	L
4L/4	-0.0002	-0.0005	-0.0006	-0.0008	-0.0005	-0.0004	0.0024	0.0096	
0L/4	-0.0012	-0.0004	0.0000	0.0006	0.0013	0.0087	0.0025	-0.0003	
1L/4	-0.0823	0.0451	0.0967	0.0965	0.0699	0.4197	0.5027	0.3256	U
2L/4	-0.0858	0.0563	0.1442	0.1374	0.0753	0.4666	0.8854	0.6497	NP6
3L/4	-0.0511	0.0360	0.0977	0.1068	0.0401	0.2685	0.6109	0.6479	L
4L/4	-0.0004	-0.0003	0.0000	0.0004	0.0005	0.0006	0.0064	0.0144	

TABLE A11 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT:	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0031	0.0056	0.0075			0.0054	0.0087		U SN6 L
1L/4	-0.1914	-0.0482	0.0135			0.1532	0.2228		
2L/4	-0.1445	-0.0376	0.0098			0.2392	0.3414		
3L/4	0.0069	0.0233	0.0135			0.1582	0.2228		
4L/4	0.0123	0.0098	0.0075			0.0066	0.0087		
0L/4	-0.0012	-0.0011	-0.0006			0.0062	-0.0003		U SA6 L
1L/4	-0.0205	0.0707	0.0947			0.3154	0.3145		
2L/4	-0.0019	0.0951	0.1400			0.3158	0.5432		
3L/4	0.0131	0.0667	0.0947			0.1657	0.3145		
4L/4	0.0005	-0.0001	-0.0006			-0.0013	-0.0003		
0L/4	-0.0015	-0.0009	-0.0006			0.0006	0.0016		U SA6*
1L/4	-0.0265	0.0748	0.0971			0.2051	0.2894		
2L/4	-0.0085	0.0977	0.1451			0.3172	0.4495		
3L/4	0.0106	0.0674	0.0971			0.2024	0.2894		
4L/4	0.0005	-0.0002	-0.0006			0.0013	0.0016		
0L/4	0.0018	0.0044	0.0064			0.0031	0.0065		U SN4 L
1L/4	-0.1604	-0.0179	0.0416			0.1876	0.2795		
2L/4	-0.0935	0.0070	0.0495			0.2981	0.4265		
3L/4	0.0484	0.0566	0.0416			0.2016	0.2795		
4L/4	0.0111	0.0086	0.0064			0.0058	0.0065		
0L/4	-0.0011	-0.0008	-0.0004			0.0038	-0.0010		U SA4 L
1L/4	-0.0233	0.0600	0.0806			0.2952	0.2938		
2L/4	-0.0030	0.0808	0.1195			0.2955	0.5143		
3L/4	0.0140	0.0581	0.0806			0.1495	0.2938		
4L/4	0.0006	0.0000	-0.0004			-0.0013	-0.0010		

TABLE A11 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF									BRIDGE TYPE
	TOP FLANGE AT X=					BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4		
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
1L/4	-0.8736	-0.4553	-0.1242	0.1516	0.3985	0.2013	0.3447	0.2803	U	
2L/4	-1.0478	-0.5740	-0.1666	0.1405	0.3859	0.3450	0.5666	0.4493	NN6	
3L/4	-0.6650	-0.3708	-0.1088	0.0973	0.2195	0.2740	0.4450	0.3496	L	
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
1L/4	-0.9027	-0.4900	-0.1612	0.1102	0.3562	0.1317	0.2285	0.1864	U	
2L/4	-1.0890	-0.6230	-0.2145	0.0871	0.3307	0.1948	0.3484	0.2870	NB6	
3L/4	-0.3746	-0.2227	-0.0827	0.0476	0.1694	0.0728	0.1388	0.2179	L	
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
1L/4	-0.0365	0.0632	0.0966	0.0767	0.0298	0.3331	0.3786	0.2395	U	
2L/4	-0.0348	0.0797	0.1430	0.1147	0.0320	0.3626	0.6649	0.4653	NA6	
3L/4	-0.0204	0.0509	0.0975	0.0898	0.0147	0.2143	0.4480	0.4679	L	
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		
1L/4	-0.0818	0.0457	0.0971	0.0965	0.0694	0.4146	0.5016	0.3257	U	
2L/4	-0.0854	0.0569	0.1448	0.1376	0.0749	0.4645	0.8770	0.6463	NP6	
3L/4	-0.0510	0.0363	0.0981	0.1071	0.0398	0.2680	0.6075	0.6395	L	
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		

TABLE A11. (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					NODES OF BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.1906	-0.0474	0.0133			0.1532	0.2226		U
2L/4	-0.1447	-0.0374	0.0104			0.2383	0.3401		SN6
3L/4	0.0046	0.0221	0.0133			0.1579	0.2226		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0202	0.0710	0.0951			0.3092	0.3146		U
2L/4	-0.0019	0.0955	0.1405			0.3154	0.5361		SA6
3L/4	0.0129	0.0668	0.0951			0.1663	0.3146		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0264	0.0756	0.0975			0.2053	0.2896		U
2L/4	-0.0085	0.0981	0.1459			0.3165	0.4485		SA6*
3L/4	0.0103	0.0675	0.0975			0.2026	0.2896		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.1596	-0.0172	0.0415			0.1879	0.2794		U
2L/4	-0.0936	0.0072	0.0500			0.2974	0.4257		SN4
3L/4	0.0467	0.0557	0.0415			0.2013	0.2794		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0231	0.0601	0.0809			0.2905	0.2937		U
2L/4	-0.0031	0.0811	0.1197			0.2948	0.5087		SA4
3L/4	0.0138	0.0582	0.0809			0.1499	0.2937		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A12 VERTICAL DEFLECTIONS W FOR GIRDER D IN MICROINCHES.

(1A) DEFLECTIONS OF TOP FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF										BRIDGE TYPE
	TOP FLANGE AT X=					BOTTOM FLANGE AT X=					
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4			
0L/4	-0.0784	-0.0528	-0.0271	-0.0015	0.0239	-0.0110	-0.0108	-0.0048			
1L/4	-2.7424	-1.5642	-0.5577	0.2762	1.0271	-0.0433	0.1038	0.1722	U		
2L/4	-3.3057	-1.9402	-0.7361	0.1943	0.9584	-0.0918	0.0657	0.1590	NN6		
3L/4	-2.1982	-1.3405	-0.5628	0.0424	0.4713	-0.1061	-0.0220	0.0563	L		
4L/4	-0.0497	-0.0383	-0.0271	-0.0161	-0.0050	-0.0140	-0.0178	-0.0117			
0L/4	-0.0834	-0.0512	-0.0255	0.0000	0.0253	-0.0066	-0.0049	-0.0009			
1L/4	-2.7316	-1.5519	-0.5422	0.2946	1.0456	0.1173	0.3348	0.3376	U		
2L/4	-3.2859	-1.9173	-0.7207	0.2156	0.9808	0.1939	0.4809	0.4605	NB6		
3L/4	-1.1852	-0.7308	-0.3196	0.0564	0.4874	0.0522	0.1505	0.3014	L		
4L/4	-0.0480	-0.0366	-0.0255	-0.0146	-0.0103	-0.0063	-0.0062	-0.0030			
0L/4	-0.0379	-0.0314	-0.0206	-0.0096	0.0013	-0.0150	-0.0083	-0.0038			
1L/4	-0.0905	0.1858	0.2883	0.2280	0.0869	0.7481	1.0025	0.6912	U		
2L/4	-0.1020	0.2382	0.4278	0.3451	0.0943	0.9550	1.6610	1.2352	NA6		
3L/4	-0.0637	0.1497	0.2920	0.2693	0.0541	0.6149	1.1825	1.0875	L		
4L/4	-0.0019	-0.0114	-0.0207	-0.0297	-0.0347	-0.0122	-0.0133	-0.0142			
0L/4	-0.0401	-0.0308	-0.0195	-0.0082	0.0030	-0.0211	-0.0086	-0.0013			
1L/4	-0.2109	0.1209	0.2873	0.2886	0.2074	0.5755	0.9137	0.7162	U		
2L/4	-0.2393	0.1680	0.4246	0.4125	0.2315	0.8000	1.4341	1.1546	NP6		
3L/4	-0.1508	0.1061	0.2905	0.3076	0.1407	0.5390	1.0342	0.9178	L		
4L/4	-0.0045	-0.0120	-0.0195	-0.0270	-0.0327	-0.0118	-0.0150	-0.0209			

TABLE A12 (CONTD.)

(1B) DEFLECTIONS OF TOP FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-0.0315	-0.0248	-0.0175			-0.0117	-0.0153		U SN6 L
1L/4	-0.6917	-0.3349	-0.1392			-0.0307	-0.0226		
2L/4	-0.5879	-0.3099	-0.1799			-0.0261	-0.0268		
3L/4	-0.1428	-0.1057	-0.1392			-0.0107	-0.0226		
4L/4	-0.0029	-0.0103	-0.0175			-0.0102	-0.0153		
0L/4	-0.0356	-0.0303	-0.0207			-0.0180	-0.0168		U SA6 L
1L/4	-0.0401	0.2100	0.2847			0.6795	0.8160		
2L/4	-0.0009	0.2867	0.4207			0.8195	1.2922		
3L/4	0.0385	0.1988	0.2847			0.4798	0.8160		
4L/4	-0.0009	-0.0109	-0.0207			-0.0135	-0.0168		
0L/4	-0.0352	-0.0307	-0.0207			-0.0146	-0.0188		U SA6* L
1L/4	-0.0321	0.2044	0.2826			0.2248	0.3159		
2L/4	0.0058	0.2841	0.4141			0.3179	0.4498		
3L/4	0.0409	0.1986	0.2826			0.2222	0.3159		
4L/4	-0.0009	-0.0109	-0.0207			-0.0125	-0.0188		
0L/4	-0.0268	-0.0199	-0.0128			-0.0114	-0.0136		U SN4 L
1L/4	-0.5390	-0.2261	-0.0321			0.0011	0.0473		
2L/4	-0.3495	-0.1235	-0.0159			0.0528	0.0892		
3L/4	0.0207	0.0237	-0.0321			0.0512	0.0473		
4L/4	0.0015	-0.0056	-0.0128			-0.0085	-0.0136		
0L/4	-0.0235	-0.0210	-0.0142			-0.0153	-0.0132		U SA4 L
1L/4	-0.0348	0.1754	0.2437			0.5679	0.7050		
2L/4	0.0020	0.2452	0.3591			0.7079	1.1150		
3L/4	0.0414	0.1743	0.2437			0.4178	0.7050		
4L/4	-0.0001	-0.0071	-0.0142			-0.0097	-0.0132		

TABLE A12 (CONTD.)

(2A) DEFLECTIONS OF BOTTOM FLANGE (NONSYMMETRIC BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF								BRIDGE TYPE
	TOP FLANGE AT X=				BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-2.7262	-1.5536	-0.5545	0.2716	1.0147	-0.0424	0.1038	0.1713	U
2L/4	-3.2947	-1.9324	-0.7321	0.1927	0.9508	-0.0899	0.0676	0.1598	NN6
3L/4	-2.1916	-1.3356	-0.5595	0.0437	0.4688	-0.1042	-0.0196	0.0578	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-2.7157	-1.5412	-0.5394	0.2896	1.0329	0.1175	0.3336	0.3359	U
2L/4	-3.2750	-1.9098	-0.7165	0.2137	0.9730	0.1931	0.4789	0.4585	NB6
3L/4	-1.1806	-0.7270	-0.3168	0.0576	0.4845	0.0532	0.1516	0.3003	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.0896	0.1877	0.2898	0.2284	0.0862	0.7493	1.0050	0.6926	U
2L/4	-0.1015	0.2398	0.4301	0.3462	0.0939	0.9558	1.6600	1.2347	NA6
3L/4	-0.0635	0.1505	0.2933	0.2707	0.0542	0.6152	1.1819	1.0853	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
0L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	
1L/4	-0.2093	0.1233	0.2888	0.2885	0.2058	0.5773	0.9154	0.7164	U
2L/4	-0.2382	0.1699	0.4269	0.4134	0.2305	0.8010	1.4337	1.1538	NP6
3L/4	-0.1502	0.1070	0.2919	0.3089	0.1404	0.5392	1.0337	0.9167	L
4L/4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	

TABLE A12 (CONTD.)

(2B) DEFLECTIONS OF BOTTOM FLANGE (SYMMETRICAL BOUNDARY CONDITIONS)

DEFL. AT	LOADS ARE APPLIED ON THE NODES OF								BRIDGE TYPE
	TOP FLANGE AT X=				BOTTOM FLANGE AT X=				
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.6874	-0.3315	-0.1384			-0.0294	-0.0211		U
2L/4	-0.5887	-0.3093	-0.1782			-0.0244	-0.0244		SN6
3L/4	-0.1483	-0.1081	-0.1384			-0.0100	-0.0211		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0393	0.2119	0.2862			0.6807	0.8185		U
2L/4	-0.0009	0.2882	0.4230			0.8209	1.2927		SA6
3L/4	0.0378	0.1992	0.2862			0.4812	0.8185		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0313	0.2065	0.2841			0.2264	0.3179		U
2L/4	0.0058	0.2856	0.4165			0.3201	0.4529		SA6*
3L/4	0.0403	0.1990	0.2841			0.2235	0.3179		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.5352	-0.2233	-0.0314			0.0026	0.0487		U
2L/4	-0.3500	-0.1231	-0.0149			0.0541	0.0911		SN4
3L/4	0.0166	0.0220	-0.0314			0.0517	0.0487		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
0L/4	0.0000	0.0000	0.0000			0.0000	0.0000		
1L/4	-0.0341	0.1767	0.2447			0.5685	0.7066		U
2L/4	0.0021	0.2461	0.3605			0.7086	1.1149		SA4
3L/4	0.0409	0.1745	0.2447			0.4188	0.7066		L
4L/4	0.0000	0.0000	0.0000			0.0000	0.0000		

TABLE A13 LONGITUDINAL REACTIONS OF GIRDERS IN LBS. FOR AN APPLIED LOAD OF 1 KIP

LOAD AT	GIRDER A		GIRDER B		GIRDER C		GIRDER D		BRIDGE TYPE	REMARKS
	X=0	X=L	X=0	X=L	X=0	X=0	X=0	X=L		
0/8L	1774.1	0.0	662.7	0.0	-680.5	0.0	-1756.3	0.0	UNN6	
0/8L	1799.7	0.0	595.1	0.0	-622.1	0.0	-1772.7	0.0	UNB6	
0/8L	100.5	0.0	-4.1	0.0	-35.2	0.0	-61.3	0.0	UNA6	
0/8L	209.8	0.0	43.3	0.0	-71.7	0.0	-181.4	0.0	UNP6	
2/8L	861.6	0.0	394.3	0.0	-392.7	0.0	-863.2	0.0	UNN6	
2/8L	897.0	0.0	313.0	0.0	-336.2	0.0	-873.8	0.0	UNB6	
2/8L	38.1	0.0	37.1	0.0	-17.6	0.0	-57.6	0.0	UNA6	
2/8L	98.4	0.0	52.5	0.0	-41.2	0.0	-109.7	0.0	UNP6	
4/8L	-36.7	0.0	103.2	0.0	-101.6	0.0	35.0	0.0	UNN6	SIMPLY SUPPORTED
4/8L	-9.1	0.0	31.8	0.0	-41.5	0.0	18.8	0.0	UNB6	
4/8L	3.2	0.0	15.9	0.0	-6.8	0.0	-12.2	0.0	UNA6	
4/8L	1.4	0.0	16.4	0.0	-8.4	0.0	-9.4	0.0	UNP6	
6/8L	-933.2	0.0	-192.4	0.0	193.1	0.0	932.5	0.0	UNN6	TOP FLANGE IS LOADED
6/8L	-909.4	0.0	-261.1	0.0	259.1	0.0	911.4	0.0	UNB6	
6/8L	-40.5	0.0	-7.7	0.0	9.8	0.0	38.4	0.0	UNA6	
6/8L	-100.9	0.0	-25.4	0.0	28.7	0.0	97.6	0.0	UNP6	
8/8L	-1831.0	0.0	-488.0	0.0	490.7	0.0	1828.2	0.0	UNN6	
8/8L	-1806.4	0.0	-558.3	0.0	557.9	0.0	1806.8	0.0	UNB6	
8/8L	-87.8	0.0	-30.0	0.0	28.3	0.0	89.6	0.0	UNA6	
8/8L	-205.1	0.0	-67.7	0.0	66.6	0.0	206.1	0.0	UNP6	
0/8L	637.6	1182.2	285.1	274.6	-304.5	-281.7	-618.3	-1175.0	USN6	BOTH ENDS PINNED.
0/8L	142.2	128.1	8.6	42.3	-48.7	-40.5	-103.8	-128.1	USA6	
0/8L	129.7	131.8	23.6	38.7	-40.7	-44.3	-114.5	-124.4	USA6*	
2/8L	180.3	720.3	167.0	126.0	-165.9	-126.0	-182.2	-719.5	USN6	
2/8L	58.2	64.1	43.2	15.4	-24.1	-16.7	-78.1	-61.9	USA6	
2/8L	69.0	63.6	30.6	15.7	-30.0	-15.8	-70.7	-62.8	USA6*	
4/8L	-268.0	268.0	23.6	-23.6	-22.6	22.6	266.3	-266.3	USN6	TOP FLANGE IS LOADED
4/8L	0.5	-0.5	14.8	-14.8	-6.3	6.3	-9.8	9.8	USA6	
4/8L	4.6	-4.6	10.6	-10.6	-9.6	9.6	-6.6	6.4	USA6*	

TABLE A13 (CONTD.)

LOAD AT	GIRDER A		GIRDER B		GIRDER C		GIRDER D		BRIDGE TYPE	REMARKS
	X=0	X=L	X=0	X=L	X=0	X=L	X=0	X=L		
2/8L	567.2	0.0	-542.7	0.0	-237.8	0.0	213.3	0.0	LNN6	SIMPLY SUPPORTED
2/8L	255.3	0.0	-129.5	0.0	-128.3	0.0	2.5	0.0	LNB6	
2/8L	740.7	0.0	-2.2	0.0	-271.7	0.0	-466.8	0.0	LNA6	
2/LB	1033.7	0.0	-354.9	0.0	-394.6	0.0	-284.2	0.0	LNP6	
4/8L	594.3	0.0	-739.1	0.0	-302.1	0.0	447.0	0.0	LNN6	BOTTOM FLANGE IS LOADED
4/8L	165.4	0.0	-185.9	0.0	-121.8	0.0	142.3	0.0	LNB6	
4/8L	679.7	0.0	133.2	0.0	-253.3	0.0	-559.6	0.0	LNA6	
4/8L	851.8	0.0	-129.0	0.0	-394.6	0.0	-328.2	0.0	LNP6	
6/8L	279.3	0.0	-512.5	0.0	-188.0	0.0	421.1	0.0	LNN6	
6/8L	-15.2	0.0	-142.9	0.0	-43.6	0.0	201.8	0.0	LNB6	
6/8L	354.0	0.0	101.5	0.0	-127.1	0.0	-328.4	0.0	LNA6	
6/8L	363.9	0.0	-24.7	0.0	-187.3	0.0	-151.9	0.0	LNP6	
2/8L	521.6	-334.2	-470.1	389.1	-183.6	161.2	135.6	-219.6	LSN6	BOTH ENDS ARE PINNED
2/8L	684.4	-176.0	-19.9	-49.4	-255.0	70.9	-412.3	157.3	LSA6	
2/8L	642.6	-529.2	-489.2	384.6	-164.0	165.4	14.9	-24.9	LSA6*	
4/8L	604.5	-604.5	-600.2	600.2	-246.4	246.4	247.0	-247.0	LSN6	
4/8L	524.8	-524.8	86.0	-86.0	-205.4	205.4	-408.5	408.5	LSA6	
4/8L	-823.5	-823.5	-610.4	610.4	-235.7	235.7	28.5	-28.5	LSA6*	

TABLE A14 TRANSVERSE REACTIONS OF GIRDERS IN LBS. FOR AN APPLIED LOAD OF 1 KIP

LOAD AT	GIRDER A		GIRDER B		GIRDER C		GIRDER D		BRIDGE TYPE	REMARKS
	X=0	X=L	X=0	X=L	X=0	X=L	X=0	X=L		
0/8L	125.6	124.3	125.7	124.4	125.7	124.4	125.6	124.3	UNN6	
0/8L	125.7	124.3	125.7	124.3	125.7	124.3	125.7	124.3	UNB6	
0/8L	168.4	3.9	356.3	6.9	301.2	6.8	152.6	3.9	UNA6	
0/8L	163.8	9.3	325.6	17.5	302.3	17.5	154.7	9.3	UNP6	
2/8L	125.4	124.5	125.5	124.7	125.5	124.7	125.3	124.5	UNN6	
2/8L	125.4	124.6	125.4	124.6	125.4	124.6	125.4	124.6	UNB6	
2/8L	126.7	45.8	239.9	86.4	241.3	86.3	127.8	45.8	UNA6	
2/8L	123.3	48.1	236.2	91.9	236.8	91.9	123.7	48.1	UNP6	
4/8L	125.1	124.8	125.2	124.9	125.2	124.9	125.1	124.8	UNN6	SIMPLY SUPPORTED
4/8L	125.1	124.9	125.1	124.9	125.1	124.9	125.1	124.9	UNB6	
4/8L	85.9	86.8	162.7	164.6	162.6	164.6	85.9	86.9	UNA6	
4/8L	85.3	86.1	163.5	165.0	163.5	165.1	85.3	86.2	UNP6	
6/8L	124.8	125.1	124.9	125.2	124.9	125.2	124.8	125.1	UNN6	TOP FLANGE IS LOADED
6/8L	124.8	125.2	124.8	125.2	124.8	125.2	124.8	125.2	UNB6	
6/8L	44.8	126.9	85.6	241.5	85.5	243.0	44.8	128.0	UNA6	
6/8L	47.3	123.5	91.2	237.5	91.2	238.0	47.3	123.9	UNP6	
8/8L	124.5	125.4	124.6	125.5	124.6	125.5	124.5	125.4	UNN6	BOTH ENDS PINNED
8/8L	124.5	125.4	124.6	125.5	124.6	125.5	124.5	125.4	UNB6	
8/8L	3.7	167.2	8.6	356.1	8.6	300.9	3.7	151.1	UNA6	
8/8L	9.3	162.7	19.2	325.1	19.1	301.7	9.3	153.5	UNP6	
0/8L	125.1	124.8	125.2	124.9	125.2	124.9	125.1	124.8	USN6	TOP FLANGE IS LOADED
0/8L	160.2	11.4	340.9	23.0	285.9	22.9	144.4	11.4	USA6	
0/8L	160.1	11.4	340.8	23.1	286.2	22.9	144.3	11.3	USA6*	
2/8L	125.0	124.9	125.1	125.0	125.1	125.0	125.0	124.9	USN6	
2/8L	122.8	49.4	232.5	94.1	233.9	94.1	123.8	49.4	USA6	TOP FLANGE IS LOADED
2/8L	122.8	49.4	232.7	94.2	233.7	94.2	123.7	49.4	USA6*	
4/8L	125.0	125.0	125.0	125.0	125.0	125.0	125.0	125.0	USN6	
4/8L	86.4	86.4	163.6	163.6	163.6	163.6	86.4	86.4	USA6	
4/8L	86.3	86.3	163.6	163.6	163.7	163.7	86.4	86.4	USA6*	

TABLE A14 (CONTD.)

LOAD AT	GIRDER A		GIRDER B		GIRDER C		GIRDER D		BRIDGE TYPE	REMARKS
	X=0	X=L	X=0	X=L	X=0	X=L	X=0	X=L		
2/8L	501.4	64.4	72.4	72.4	72.4	72.4	72.3	72.3	LNN6	SIMPLY SUPPORTED
2/8L	501.2	64.3	72.4	72.4	72.4	72.4	72.4	72.4	LNB6	
2/8L	100.5	70.3	198.7	135.2	190.8	136.3	97.8	70.4	LNA6	
2/8L	104.7	71.9	194.1	136.6	186.3	137.1	98.6	70.8	LNP6	
4/8L	199.1	199.2	100.3	100.4	100.2	100.3	100.2	100.2	LNN6	BOTTOM FLANGE IS LOADED
4/8L	198.9	199.0	100.3	100.4	100.3	100.4	100.3	100.4	LNB6	
4/8L	54.4	111.6	108.3	222.9	110.1	223.9	56.3	112.7	LNA6	
4/8L	55.3	110.0	112.8	219.1	114.2	217.9	59.3	111.6	LNP6	
6/8L	64.3	501.5	72.3	72.5	72.3	72.4	72.3	72.4	LNN6	
6/8L	64.2	501.3	72.4	72.5	72.4	72.5	72.3	72.5	LNB6	
6/8L	26.4	142.9	50.3	286.6	51.7	277.0	26.7	138.4	LNA6	
6/8L	30.9	146.2	57.8	275.0	59.4	263.7	30.6	136.4	LNP6	
2/8L	501.4	64.4	72.4	72.4	72.4	72.4	72.4	72.4	LSN6	
2/8L	111.3	60.4	218.9	114.0	211.0	115.1	102.6	60.7	LSA6	
2/8L	485.6	39.2	103.6	89.5	94.1	84.8	55.3	47.5	LSA6*	
4/8L	199.1	199.1	100.3	100.3	100.3	100.3	100.3	100.3	LSN6	BOTH ENDS ARE PINNED
4/8L	84.2	84.2	164.0	164.0	165.7	165.7	86.1	86.1	LSA6	
4/8L	170.3	170.3	134.0	134.0	124.5	124.5	71.2	71.2	LSA6*	

TABLE A15 VERTICAL REACTIONS OF GIRDERS IN LBS. FOR AN APPLIED LOAD OF 1 KIP

LOAD AT	GIRDER A		GIRDER B		GIRDER C		GIRDER D		BRIDGE TYPE	REMARKS
	X=0	X=L	X=0	X=L	X=0	X=L	X=0	X=L		
0/8L	-24.8	-244.2	75.2	-14.0	-79.7	13.3	29.3	244.9	UNN6	
0/8L	34.1	-234.9	-35.6	-41.8	-34.9	41.2	36.5	235.5	UNB6	
0/8L	-266.1	-9.8	28.8	-2.2	22.3	1.4	215.0	10.7	UNA6	
0/8L	-235.6	-25.0	15.9	-4.8	5.2	3.5	214.5	26.3	UNP6	
2/8L	-83.9	-187.6	60.9	1.7	-60.2	-1.7	83.2	187.5	UNN6	
2/8L	-65.8	-179.0	14.8	-24.7	-22.2	25.3	73.2	178.3	UNB6	
2/8L	-182.0	-67.6	-2.7	-5.8	9.6	5.2	175.1	68.2	UNA6	
2/8L	-175.3	-76.2	4.0	-4.3	0.0	4.1	171.4	76.3	UNP6	
4/8L	-138.4	-130.9	38.7	17.6	-38.3	-17.5	138.0	130.7	UNN6	SIMPLY SUPPORTED
4/8L	-125.1	-122.8	0.1	-7.3	-1.0	8.2	126.0	121.9	UNB6	
4/8L	-122.4	-123.6	-5.3	-8.6	6.6	8.0	121.1	124.1	UNA6	
4/8L	-124.5	-125.3	-0.2	-3.4	1.8	3.7	122.8	125.0	UNP6	TOP FLANGE IS LOADED
6/8L	-193.1	-74.6	16.6	34.5	-16.5	-34.1	192.9	74.1	UNN6	
6/8L	-180.9	-64.2	-19.9	7.1	20.4	-10.3	180.5	67.4	UNB6	
6/8L	-66.3	-180.9	-4.6	-7.6	4.3	10.5	66.6	177.9	UNA6	
6/8L	-75.3	-174.8	-3.0	-0.1	3.3	2.1	75.0	172.7	UNP6	
8/8L	-247.5	-15.8	-5.5	47.5	4.8	-49.8	248.2	18.1	UNN6	
8/8L	-235.8	35.5	-40.6	-43.5	40.2	-22.1	236.2	30.1	UNB6	
8/8L	-11.1	-269.3	-2.9	35.1	1.9	24.0	12.1	210.2	UNA6	
8/8L	-26.4	-236.8	-5.5	19.5	4.2	6.5	27.8	210.8	UNP6	
0/8L	-21.0	-250.0	67.3	-0.6	-72.1	0.7	25.8	249.9	USN6	
0/8L	-243.8	-31.9	32.6	-6.5	18.4	5.5	192.7	32.9	USA6	
0/8L	-249.0	-32.5	38.8	-6.1	21.7	6.2	188.5	32.3	USA6*	BOTH ENDS PINNED
2/8L	-81.7	-193.7	58.3	18.1	-55.6	-18.0	81.0	193.6	USN6	
2/8L	-171.3	-78.8	-0.8	-6.7	7.7	6.5	164.4	79.0	USA6	
2/8L	-166.5	-79.1	-6.6	-6.3	5.1	6.5	168.0	79.0	USA6*	
4/8L	-137.7	-137.7	37.1	37.1	-36.8	-36.8	137.3	137.3	USN6	TOP FLANGE IS LOADED
4/8L	-123.7	-123.7	-5.5	-5.5	6.8	6.8	122.5	122.5	USA6	
4/8L	-123.3	-123.3	-5.9	-5.9	6.1	6.1	123.1	123.1	USA6*	

TABLE A15 (CONTD.)

LOAD AT	GIRDER A		GIRDER B		GIRDER C		GIRDER D		BRIDGE TYPE	REMARKS
	X=0	X=L	X=0	X=L	X=0	X=L	X=0	X=L		
2/8L	162.5	30.8	-182.5	-43.1	-75.3	-53.6	95.3	65.8	LNN6	SIMPLY SUPPORTED
2/8L	65.4	-7.4	-52.5	-4.8	-44.0	-15.5	31.0	27.7	LNB6	
2/8L	133.4	-85.9	-30.2	4.8	-96.2	4.6	-7.0	76.5	LNA6	
2/8L	250.3	-93.0	-177.7	15.2	-145.6	-1.1	73.0	79.0	LNP6	
4/8L	172.6	84.2	-204.0	-86.1	-109.5	-80.7	140.9	82.6	LNN6	BOTTOM FLANGE IS LOADED
4/8L	48.9	19.3	-47.9	-13.6	-50.7	-30.9	49.7	25.2	LNB6	
4/8L	79.1	-94.7	36.0	19.6	-52.4	-12.1	-62.7	87.2	LNA6	
4/8L	121.5	-76.8	-14.1	16.9	-98.1	-41.6	-9.3	101.4	LNP6	
6/8L	89.5	95.2	-118.8	-89.8	-77.7	-59.2	107.0	53.9	LNN6	
6/8L	10.0	39.7	-24.2	-21.5	-28.5	-29.3	42.7	11.0	LNB6	
6/8L	37.0	-20.0	26.0	-12.2	-20.2	-58.3	-42.7	90.3	LNA6	
6/8L	36.3	60.4	17.0	-92.6	-36.0	-102.9	-17.3	135.1	LNP6	
2/8L	150.5	74.9	-170.4	-103.8	-63.4	-64.3	83.4	93.1	LSN6	BOTH ENDS ARE PINNED
2/8L	104.1	-55.1	-35.3	9.6	-91.1	-4.3	22.4	49.8	LSA6	
2/8L	147.0	93.1	-189.5	-124.7	-45.8	-46.2	88.2	77.9	LSA6*	
4/8L	153.8	153.8	-184.9	-184.9	-91.7	-91.7	122.8	122.8	LSN6	
4/8L	-1.7	-1.7	21.8	21.8	-38.4	-38.4	18.3	18.3	LSA6	
4/8L	164.6	164.6	-213.6	-213.6	-66.6	-66.6	115.6	115.6	LSA6*	

TABLE A16 COEFFICIENTS OF COMPOSITE GIRDER MOMENTS CM#10

LOAD AT X=L*	GIRD A	QUARTER GIRD B	SPAN GIRD C	GIRD D	GIRD A	MID GIRD B	SPAN GIRD C	GIRD D	THREE GIRD A	QUARTER GIRD B	SPAN GIRD C	BRIDGE GIRD D	BRIDGE TYPE
0.0	-.978	-.319	.319	.978	-.722	-.241	.238	.725	-.428	-.117	.115	.430	UNN6
.25	-.573	-.181	.181	.572	-.461	-.147	.148	.461	-.300	-.071	.071	.300	UNN6
.50	-.163	-.046	.045	.163	-.196	-.056	.056	.195	-.169	-.026	.026	.169	UNN6
.75	.249	.091	-.091	-.249	.074	.033	-.033	-.075					UNN6
1.0	.663	.227	-.230	-.659	.349	.119	-.124	-.344					UNN6
0.0	-.973	-.329	.329	.974	-.718	-.251	.247	.721					UNB6
.25	-.561	-.201	.190	.573	-.458	-.157	.159	.455					UNB6
.50	-.160	-.054	.058	.157	-.183	-.080	.066	.196					UNB6
.75	.252	.080	-.077	-.255	.078	.021	-.080	-.080	-.018	-.021	.005	.033	UNB6
1.0	.666	.215	-.216	-.665	.353	.106	-.109	-.350	.082	-.031	-.035	-.107	UNB6
0.0	-.030	-.021	.012	.039	-.019	-.013	.006	.026	-.010	-.005	.003	.013	UNA6
.25	-.033	-.006	.011	.023	-.019	-.007	.006	.020	-.009	-.005	.003	.011	UNA6
.50	-.007	-.003	.003	.010	-.016	-.008	.003	.003	-.005	-.003	.002	.006	UNA6
.75	.016	.003	-.005	-.014	.008	.002	-.003	-.007	-.006	.011	.002	-.007	UNA6
1.0	.041	.013	-.014	-.040	.030	-.004	-.011	-.023	.022	-.009	-.006	-.007	UNA6
0.0	-.087	-.036	.032	.091	-.054	-.025	.019	.060	-.022	-.012	.008	.031	UNP6
.25	-.057	-.013	.018	.052	-.037	-.012	.012	.036	-.014	-.007	.007	.019	UNP6
.50	-.008	-.004	.003	.009	-.013	-.004	.002	.007	.005	-.001	.002	.006	UNP6
.75	.042	.012	-.013	-.041	.025	-.009	-.008	-.025	.008	.011	-.004	-.012	UNP6
0.0	-.290	-.086	.087	.290	-.041	-.012	.009	.044	.269	.096	-.100	-.265	USN6
.25	-.160	-.040	.041	.159	-.052	-.010	.011	.052	.120	.050	-.050	-.120	USN6
.50	-.022	.004	-.004	.023	-.057	-.009	.010	.056	-.022	.004	-.004	.023	USN6
0.0	-.030	-.021	.012	.040	-.007	-.004	-.003	.001	.042	.012	-.014	-.039	USA6
.25	-.033	-.000	.011	.023	-.006	-.002	.002	.007	.016	.003	-.005	-.014	USA6
.50	-.007	-.001	-.003	.010	-.017	.007	.004	.007	-.007	-.005	.003	.010	USA6
0.0*	.035	-.014	.014	.036	.005	-.003	-.000	-.001	.039	.011	-.015	-.038*	USA6
.25	-.027	-.008	.009	.027	-.007	-.001	.002	.007	.016	.004	-.004	-.015*	USA6
.50	-.009	-.003	.003	.009	-.012	-.000	.001	.011	.005	-.003	.003	.009*	USA6
.25	1.466	.505	.112	-.209	.894	.438	.075	-.158	.351	.240	.046	-.088	**
.50	.813	.524	.145	-.232	2.005	.619	.070	-.195	.772	.393	.036	-.117	**
.75	.365	.303	.101	-.144	.922	.410	.049	-.131	1.362	.348	-.001	-.081	**
.25	.507	.950	.306	.112	.439	.418	.315	.078	.200	.168	.168	.048	***
.50	.525	.311	.269	.144	.621	1.291	.513	.074	.331	.467	.349	.039	***
.75	.303	.095	.126	.100	.412	.447	.339	.052	.299	1.157	.367	.001	***

TABLE A16 (CONTD.)

LOAD AT X=L*	QUARTER SPAN BRIDGE	QUARTER SPAN				MID SPAN				THREE QUARTER SPAN				BRIDGE TYPE
GIRD A	GIRD B	GIRD C	GIRD D	GIRD A	GIRD B	GIRD C	GIRD D	GIRD A	GIRD B	GIRD C	GIRD D	GIRD D		
.25	.030	-.043	.000	.012	.190	-.171	-.080	.061	.114	-.105	-.080	.071	LNN6	
.50	.075	.002	-.023	-.054	.330	-.237	-.132	.039	.234	-.185	-.127	.078	LNN6	
.75	.249	.091	-.091	-.249	.074	.033	-.033	-.075					LNN6	
.25	.010	.002	-.016	.004	.068	.003	-.046	-.025					LNB6	
.50	.059	.037	-.033	-.064	.153	.017	-.085	-.085					LNB6	
.75	.076	.046	-.030	-.091	.136	.025	-.069	-.092	.084	.001	-.058	-.043	LNB6	
.25	.221	-.067	-.076	-.078	.071	.054	-.029	-.096	.028	.030	-.005	-.053	LNA6	
.50	-.044	.027	.014	.003	.438	-.004	-.154	-.281	.156	.079	-.058	-.178	LNA6	
.75	-.046	.012	.021	.013	.143	.079	-.054	-.168	.459	-.021	-.163	-.275	LNA6	
.25	.319	-.230	-.091	.001	.090	.035	-.061	-.064	.014	.048	.010	-.051	LNP6	
.50	-.039	.059	.003	-.023	.604	-.210	-.216	-.179	.166	.050	-.109	-.141	LNP6	
.25	.030	-.060	-.005	.032	.153	-.148	-.053	.045	.013	-.004	-.014	.004	LSN6	
.50	.027	-.041	-.014	.025	.226	-.218	-.074	.064	.027	-.041	-.014	.025	LSN6	
.25	.222	-.066	-.076	-.077	-.038	.043	-.017	-.061	-.042	.009	.020	.015	LSA6	
.50	-.042	.026	.014	.004	.346	-.035	-.122	-.187	-.042	.026	.014	.004	LSA6	
.25	.050	-.060	-.005	.012	.179	-.143	-.058	.019	-.010	-.012	-.007	.007*	LSA6	
.50	.038	-.047	-.009	.014	.262	-.211	-.081	.027	.020	-.047	-.009	.014*	LSA6	

* DIAPHRAGMS PROVIDED ONLY AT END SECTIONS.
 ** VERTICAL LOADS ON GIRDER A OF NONSYMMETRICALLY SUPPORTED NO DIAPHRAGM BRIDGE
 *** VERTICAL LOADS ON GIRDER B OF NONSYMMETRICALLY SUPPORTED NO DIAPHRAGM BRIDGE

TABLE A17 TRANSVERSE FORCE IN LBS.* RESISTED BY DIAPHRAGMS BETWEEN GIRDERS A AND B FOR AN APPLIED LOAD OF 1 KIP.

DIAP. AT X=	LOADS ARE APPLIED ON THE NODES OF TOP FLANGE AT X=					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-373.1	-239.5	-163.0	-85.8	-8.1	-193.8	-107.5		
1L/4	9.9	-12.2	2.9	0.4	-1.1	473.4	-45.5		U
2L/4	0.9	3.7	-9.0	4.4	-0.3	-45.1	446.4		NA6
3L/4	-0.9	0.6	3.9	-10.2	6.4	-6.8	-54.5		L
4L/4	-0.7	-86.7	-164.9	-240.7	-372.4	-135.0	-220.5		
0L/4	-357.7	-232.2	-164.0			-214.0	-163.4		
1L/4	10.0	-12.2	2.8			473.4	-45.0		U
2L/4	1.0	3.6	-9.5			-44.6	450.0		SA6
3L/4	-0.8	0.2	2.8			-5.5	-45.0		L
4L/4	-22.5	-94.2	-164.0			-114.5	-163.4		
0L/4	-357.8	-232.4	-164.0			-104.6	-132.8		U
4L/4	-22.7	-94.2	-164.0			-88.0	-132.8		SA6*L
0L/4	-333.5	-237.4	-164.5	-91.6	-18.8	-191.8	-113.0	-58.1	
1L/4	1.8	-6.1	0.1	-0.3	-0.7	644.3	-42.5	-20.6	U
2L/4	1.9	2.0	-3.1	2.4	1.5	-41.7	601.7	-55.5	NP6
3L/4	-0.4	-0.2	0.5	-5.4	1.6	-21.0	-57.5	-568.9	L
4L/4	-17.6	-92.6	-166.0	-238.3	-332.6	-137.4	-219.7	-273.2	
0L/4	-55.2	-19.6	-13.8	-12.8	-12.4	-22.7	-14.6	-14.9	
1L/4	-3.5	-18.0	-12.3	-5.9	-5.9	-70.7	-89.6	-59.2	U
2L/4	0.3	-6.1	-13.3	-5.8	1.8	-108.1	-151.6	-111.7	NB6
3L/4	-3.3	-3.8	-9.5	1.7	-3.3	-77.0	-121.9	-91.4	L
4L/4	-15.5	-14.7	-14.5	-18.6	-52.7	-11.9	-19.7	-17.6	

* DIAPHRAGMS PROVIDED ONLY AT END SECTIONS.

TABLE A18 TRANSVERSE FORCE IN LBS.* RESISTED BY DIAPHRAGMS BETWEEN GIRDERS B AND C FOR AN APPLIED LOAD OF 1 KIP.

DIAP. AT X=	TRANSVERSE LOAD APPLIED ON					BRIDGE TYPE			
	TOP FLANGE AT X=	BOTTOM FLANGE AT X=			BRIDGE TYPE				
0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	TYPE	
0L/4	-318.2	-252.4	-170.3	-89.1	-8.3	-196.9	-112.7	U	
1L/4	-4.3	-2.0	-1.4	-0.5	0.1	325.6	16.4	NA6	
2L/4	1.0	-0.5	3.7	-0.9	1.1	16.1	329.2	L	
3L/4	0.2	-0.9	-2.3	2.2	-3.3	-2.9	16.2		
4L/4	-7.5	-90.4	-172.3	-253.5	-314.9	-140.7	-229.6		
0L/4	-302.0	-244.6	-171.3			-218.3	-171.6	U	
1L/4	-4.3	3.5	-1.4			325.6	16.4	SA6	
2L/4	1.0	-0.5	3.9			16.5	374.6	L	
3L/4	0.0	-0.6	-1.4			-2.0	16.4		
4L/4	-23.5	-98.2	-171.3			-119.7	-171.6		
0L/4	-302.2	-244.8	-171.4			-98.6	-130.6	U	
4L/4	-23.5	-98.4	-171.4			-88.9	-130.6	SA6*L	
0L/4	-311.8	-241.7	-167.0	-92.8	-18.9	-190.8	-115.7	-59.5	U
1L/4	-3.3	0.4	-1.4	-1.0	-0.3	238.7	24.2	-0.9	NP6
2L/4	1.1	0.6	2.3	0.4	1.1	25.6	248.4	28.5	L
3L/4	-0.1	-1.0	-1.9	-0.3	-3.1	-1.7	26.1	-	
4L/4	-18.1	-94.0	-168.6	-242.5	-310.3	-139.4	-221.9	-270.0	
0L/4	-98.1	-23.6	-23.5	-21.9	-20.9	-1.2	-8.3	-8.5	U
1L/4	-58.6	-54.3	-54.7	-60.7	-62.3	86.2	87.9	40.3	NB6
2L/4	-33.8	-31.6	-53.3	-35.7	-41.9	121.9	179.2	122.1	L
3L/4	-64.6	-65.6	-62.3	-82.5	-66.6	43.1	93.6	93.8	
4L/4	-22.1	-21.8	-22.0	-21.1	-94.9	-9.1	-9.6	-2.6	

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* DIAPHRAGMS PROVIDED ONLY AT END SECTIONS.

TABLE A19 TRANSVERSE FORCE IN LBS.* RESISTED BY DIAPHRAGMS BETWEEN GIRDERS C AND D FOR AN APPLIED LOAD OF 1 KIP.

DIAP. AT X=	TRANSVERSE LOAD APPLIED ON					BOTTOM FLANGE AT X=			BRIDGE TYPE
	0L/4	1L/4	2L/4	3L/4	4L/4	1L/4	2L/4	3L/4	
0L/4	-286.3	-242.4	-163.1	-85.4	-8.0	-193.0	-109.6		U NA6 L
1L/4	-7.3	6.3	-2.2	-0.8	0.5	191.3	35.7		
2L/4	-0.3	-1.5	5.5	-0.5	0.6	36.2	213.4		
3L/4	0.4	-0.6	-2.4	5.5	-4.9	6.5	43.3		
4L/4	-7.0	-86.7	-165.0	-243.8	-285.1	-135.4	221.0		
0L/4	-271.0	-235.0	-164.0			-213.3	-165.6		U SA6 N
1L/4	-7.2	6.4	-2.1			181.2	34.8		
2L/4	-0.7	-1.4	5.7			35.3	209.4		
3L/4	0.2	-0.6	-2.1			4.7	34.8		
4L/4	-22.5	-94.1	-164.0			-115.2	-165.6		
0L/4	-271.0	-235.3	-164.2			-106.6	-137.5		U SA6*L
4L/4	-22.5	-94.2	-164.2			-91.8	-137.5		
0L/4	-298.9	-238.5	-164.6	-91.6	-18.8	-192.1	-115.6	-59.5	U NP6 L
1L/4	-3.6	1.2	-1.0	-0.8	-0.4	94.0	33.4	14.4	
2L/4	0.1	1.0	2.9	0.9	0.8	35.2	125.5		
3L/4	-0.5	-0.7	-1.0	1.0	-2.9	14.2	44.3	112.2	
4L/4	-17.7	-92.6	-166.2	-239.5	-297.8	-137.2	-218.5	-267.9	
0L/4	-67.4	-10.3	-13.1	-13.1	-12.1	0.4	-1.1	-2.4	U NB6 L
1L/4	-9.1	-25.2	-1.4	-4.4	-5.2	28.4	38.3	27.5	
2L/4	-2.9	4.6	-14.8	6.9	1.2	57.9	82.9	61.2	
3L/4	-3.3	-1.4	3.2	-19.2	-3.3	42.8	63.7	45.9	
4L/4	-15.2	-15.0	-14.0	-10.0	-65.8	-5.1	-5.3	-2.8	

* DIAPHRAGMS PROVIDED ONLY AT END SECTIONS.

TABLE A20 REACTIONS OF 4 FEET DEEP GIRDERS IN LBS. FOR AN APPLIED LOAD OF 1 KIP.

LOAD AT	GIRDER A		GIRDER B		GIRDER C		GIRDER D		BRIDGE TYPE	REMARKS
	X=0	X=L	X=0	X=L	X=0	X=L	X=0	X=L		
0/8L	121.1	99.7	-5.4	33.0	-41.7	-30.9	-75.7	-100.0	USA4	LONGI-TUDINAL REACTIONS
0/8L	534.4	858.1	220.4	188.3	-246.6	-197.7	-508.6	-848.4	USN4	
2/8L	37.1	53.3	42.3	11.0	-17.7	-13.3	-62.8	-49.9	USA4	
2/8L	176.2	507.9	143.0	78.5	-141.3	-78.8	-178.8	-506.7	USN4	
4/8L	-6.6	6.6	16.0	-16.0	-3.1	3.1	-7.3	7.3	USA4	
4/8L	-165.6	165.6	34.9	-34.9	-33.1	33.1	162.9	-162.9	USN4	
2/8L	561.9	-132.7	-64.2	-34.0	-211.1	57.2	-288.3	111.3	LSA4	
2/8L	599.6	-288.2	-491.3	353.1	-175.2	143.6	72.5	-214.2	LSN4	
4/8L	414.5	-414.5	46.2	-46.2	-169.7	169.7	-292.8	292.8	LSA4	
4/8L	624.3	-624.3	-583.6	583.6	-229.5	229.5	196.7	-196.7	LSN4	
0/8L	155.6	11.6	362.6	23.8	277.1	23.7	134.0	11.6	USA4	TRANSVERSE REACTIONS
0/8L	125.2	124.7	125.3	124.8	125.3	124.8	125.2	124.7	USN4	
2/8L	120.8	48.8	233.6	95.2	235.7	95.2	121.9	48.8	USA4	
2/8L	125.1	124.8	125.2	124.9	125.2	124.9	125.1	124.8	USN4	
4/8L	-85.0	85.0	164.8	164.8	165.0	165.0	85.3	85.3	USA4	
4/8L	124.9	124.9	125.1	125.1	125.1	125.1	124.9	124.9	USN4	
2/8L	108.8	60.1	215.9	115.7	212.6	116.6	109.9	60.2	LSA4	
2/8L	401.9	39.4	93.1	93.2	93.2	93.1	93.1	93.0	LSN4	
4/8L	83.3	83.3	164.3	164.3	166.6	166.6	85.8	85.8	LSA4	
4/8L	119.9	119.9	126.7	126.7	126.7	126.7	126.6	126.6	LSN4	
0/8L	-177.2	-21.9	40.5	-4.6	12.1	3.3	124.6	23.2	USA4	VERTICAL REACTIONS
0/8L	-16.2	-170.1	53.1	10.3	-58.1	-9.9	21.2	169.7	USN4	
2/8L	-119.2	-54.1	5.4	-3.5	2.8	3.3	111.0	54.3	USA4	
2/8L	-58.2	-132.5	48.4	22.6	-47.7	-22.5	57.4	132.3	USN4	
4/8L	-85.2	-85.3	-1.1	-1.1	3.0	3.0	83.4	83.4	USA4	
4/8L	-95.3	-95.3	35.5	35.5	-35.0	-35.0	94.9	94.9	USN4	
2/8L	88.2	-38.5	-44.8	8.1	-71.2	-4.3	27.9	34.7	LSA4	
2/8L	156.1	45.1	-174.2	-73.2	-51.7	-57.2	69.8	85.3	LSN4	
4/8L	2.1	2.1	12.5	12.5	-31.2	-31.2	16.6	16.6	LSA4	
4/8L	133.4	133.4	-160.4	-160.4	-79.3	-19.3	106.3	106.3	LSN4	

LITERATURE CITED

- (1) Lightfoot, E. and Sawko, F., "Grid Frameworks Resolved by Generalized Slope-Deflection", Engineering, Vol. 187, pp 18-20, 1959.
- (2) Lightfoot, E. and Sawko, F., "The Analysis of Grid Frameworks and Floor Systems by the Electronic Computers", Structural Engineer, Vol. 38, 1960.
- (3) Hendry, A. W. and Jaeger, L. G., "The Analysis of Grid Frameworks and Related Structures", Chatto and Windus, London, 1958.
- (4) Hetenyi, M., "A Method of Calculating Grillage Beams", S. Timoshenko 60th Anniversary Volume, Macmillian, New York, 1938.
- (5) Pippard, A. J. S. and DeWaele, J. P. A., "The Loading of Interconnected Bridge Girders", Journal of the Institution of Civil Engineers, Vol. 10, No. 1, Nov., 1938.
- (6) Davis, R. E., Kozak, J. J. and Scheffey, C. F., "Structural Behavior of a Concrete Box Girder Bridge", Highway Research Record, No. 76, 1965, p.32.
- (7) Newmark, N. M., "A Distribution Procedure for the Analysis of Slabs Continuous Over Flexible Beams", Bulletin No. 304, Univ. of Illinois, Urbana, Illinois, July 1938.
- (8) Huber, M. T., "Die Theorie der Kreuzweise bewehrten Eisenbetonplatten", Der Bauingenieur, V. 4, 1923, p.354.
- (9) Guyan, Y., "Calcul des Ponts Larges a Poutres Multiples Solidarisees par des Entretoises", Annales des Ponts et Chaussees, No. V, Septembre -Octobre, 1946.
- (10) Massonnet, Ch., "Contribution au calcul des ponts a poutres multiples", Annales des Travaux Publics de Belgique, Juin, Octobre, Decembre, 1950.
- (11) Massonnet, Ch., "Methode de calcul des ponts a poutres multiples tenant compte de leur resistance a la torsion", Publications International Association for Bridge and Structural Engineering, Vol. 10, 1950.
- (12) Massonnet, Ch., "Plaques et Coques Cylindriques Orthotropes a Neryures Dissymetriques", Publications International Association for Bridge and Structural Engineering, No. 19, 1959.
- (13) Cornelius, W., "Die Berechnung der ebenen Flachentragwerke mit Hilfe der Theorie der Orthogonal-anisotropen Platte", Der Stahlbau, Nos. 21, 43, 60, 1952.

LITERATURE CITED -- Cont..

- (14) Pflüger, A., "Zum Beulproblem der Anisotropen Rechteckplatte", Ingenieur - Archiy., No. 16, 1947.
- (15) Pflüger, A., "Die Orthotrope Platte mit Hohlsteifen", Österreichisches Ingenieur - Archiy., V. 2, 1955.
- (16) Trenks, K., "Beitrag zur Berechnung Orthogonal Anisotropen Rechteckplatten", Bauingenieur, No. 10, 1954.
- (17) Glencke, E., "Die Grundgleichungen für die Orthotrope Platte mit Exzentrischen Steifen", Der Stahlbau, No. 6, 1955.
- (18) Chu, K. H. and G. Krishnamoorthy, "Use of Orthotropic Plate Theory in Bridge Design", Journal of the Structural Division, ASCE, Vol. 88, Nos. ST3, June 1962.
- (19) V. Vitols, Clifton, R. J., and T. Au, "Analysis of Composite Beam Bridges by Orthotropic Plate Theory", Journal of the Structural Division, ASCE, Vol. 89, No. ST4, August 1963.
- (20) Clifton, R. J., J. C. L. Chang, and Tung Au, "Analysis of Orthotropic Plate Bridges", Journal of the Structural Division, ASCE, Vol. 89, No. ST5, Proc. Paper 3675, October, 1963.
- (21) Adotte, G. D., "Second Order Theory in Orthotropic Plates", Journal of the Structural Division, ASCE, Vol. 93, No. ST5, October 1967.
- (22) Heins, C. P., Jr., and Charles T. G. Looney, "Bridge Analysis Using Orthotropic Plate Theory", Journal of the Structural Division, ASCE, Vol. 94, ST2, Feb. 1968.
- (23) Troitsky, M. S., "Orthotropic Bridges Theory and Design", The James F. Lincoln Arc Welding Foundation, Cleveland, Ohio, August 1967.
- (24) "Design Manual for Orthotropic Steel Plate Deck Bridges", American Institute of Steel Construction, New York, 1963.
- (25) Zienkiewicz, O. C. and Cheung, Y. K., "The Finite Element Method in Structural and Continuum Mechanics", McGraw Hill Publishing Company Ltd. 1967.
- (26) Prezemieniecki, J. S., "Theory of Matrix Structural Analysis", McGraw Hill Publishing Company, 1968.
- (27) Clough, R. W., "The Finite Element Method in Plane Stress Analysis", 2nd Conference on Electronic Computation, Sept. 1960.

LITERATURE CITED - Cont..

- (28) B. Graeijs de Yeubeke, "Displacement and Equilibrium Models in the Finite Element Methods", Stress Analysis, ed. O. C. Zienkiewicz and G. S. Holister, Wiley, 1965.
- (29) Pian, T. H. H., "Element Stiffness Matrices for Boundary Compatibility and for Prescribed Boundary Stresses", Proc. Conf. Matrix Methods Struc. Mech., Air Force Inst. of Technology, Wright Patterson Air Force Base, Oct. 1965.
- (30) Clough, R. W., and Tocher, J. L., "Finite Element Stiffness Matrices for Analysis of Plate Bending", Proceedings of the Conference on Matrix Methods of Structural Mechanics, Wright-Patterson Air Force Base, Ohio, 1965.
- (31) Zienkiowhy, O. C., and Holister, G. S., "Stress Analysis", John Wiley and Sons Ltd., 1965.
- (32) Tocher, J. L. and Hartz, B. J., "Higher Order Plate Element for Plane Stress", Jour. of the Engineering Mechanics Division, ASCE, Vol. 93, No. EM4, August 1967.
- (33) Bergan, P. G., "Plane Stress Analysis Using the Finite Element Method, Triangular Element with Six Parameters in Each Node", Division of Structural Mechanics, The Technical University of Norway, Trondheim, Norway, Oct. 1967.
- (34) Turner, M. J., Cough, R. W., Martin, H. C. and Topp, L. J., "Stiffness and Deflection Analyses of Complex Structures", Journal of Aeronautical Sciences, Vol. 23, No. 9, Sept. 1956.
- (35) Melosh, R. J., "Basis for Derivations of Matrices for the Direct Stiffness Method", Journal of the American Institute of Aeronautics and Astronautics, Vol. 1, No. 7, July 1963, pp. 1631-1637.
- (36) Hooley, R. F., and Hibbert, P. D., "Bounding Plane Stress Solutions by Finite Elements", Journal of the Structural Division, ASCE, Vol. 92, No. ST1, Proc. Paper 4663, Feb. 1966.
- (37) Wilson, L. B., "Solution of Certain Large Sets of Equations on Pegasus Using Matrix Methods", The Computer Journal, Vol. 2, 1959, p. 130.
- (38) Asplund, S. O., "Inversion of Band Matrices", 2nd ASCE Conf. Electronic Computations, 1960, pp. 513-22.
- (39) Livesley, R. K., "The Analysis of Large Structural Systems", The Computer Journal, Vol. 3, 1960, p. 34.

LITERATURE CITED - Cont.

- (40) Clough, R. W., E. L. Wilson and I. P. King, "Large Capacity Multi-story Frame Analysis Programs", Journal of the Structural Division, ASCE, Vol. 89, No. ST4, Proc. Paper 3592, Aug. 1963, pp. 179-204.
- (41) Gatewood, R. E. and Norik Ohanian, "Examples of Solution Accuracy in Certain Large-Simultaneous Equation Systems", Proc. Conf. on Matrix Methods in Structural Mechanics, Air Force Inst. of Technology, Wright Patterson Air Force Base, Oct. 1965.
- (42) Przemieniecki, J. S., "Matrix Structural Analysis of Substructures", AIAA Journal, Vol. 1, Jan. 1963, pp. 138-147.
- (43) Livesley, R. K., "Matrix Methods of Structural Analysis", New York, Pergamon Press, Macmillan, 1964.
- (44) William Weaver Jr., "Computer Programs for Structural Analysis", D. Van Nostrand Company, Inc. 1967.
- (45) Rubinstein, Moshe F., and Wikholm, Duane E., "Analysis by Group Iteration Using Substructures", Jour. of the Structural Divn., ASCE, Vol. 94, No. ST2, Proc. Paper 5779, Feb. 1968, pp. 363-375.
- (46) Gustafson, W. C., "Analysis of Eccentrically Stiffened Skew Plate Structures", Ph. D. Thesis, Illinois (Urbana), 1966.
- (47) Vlasov, V. Z., "Thin-walled Elastic Beams", OTS 61-11400, Office of Technical Services, U. S. Dept. of Commerce, Israel Program for Scientific Translations, Catalogue No. 428, Wash. D. C., 1959.
- (48) Kuang-Han Chu and Anatole Longinow, "Torsion in Sections with Open and Closed Parts", Journal of the Structural Division, ASCE, Vol. 93, No. ST6, December, 1967.

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