Retrospective Theses and Dissertations

Iowa State University Capstones, Theses and Dissertations

1-1-2001

Finite element analysis of longitudinal glulam timber deck and glulam timber girder bridges

Anil Varughese Kurian
Iowa State University

Follow this and additional works at: https://lib.dr.iastate.edu/rtd

Part of the Engineering Commons

Recommended Citation

Finite element analysis of longitudinal glulam timber deck and glulam timber girder bridges

by

Anil Varughese Kurian

A thesis submitted to the graduate faculty

in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

Major: Civil Engineering (Structural Engineering)

Major Professors: Terry J. Wipf and Fouad Fanous

Iowa State University

Ames, Iowa

2001

Graduate College
Iowa State University

This is to certify that the Master's thesis of

Anil Varughese Kurian

has met the thesis requirements of Iowa State University

Signatures have been redacted for privacy

Signatures have been redacted for privacy

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

1.1 General

The three most common materials used for bridge construction are wood, steel and reinforced concrete. Although in the 20th century, concrete and steel replaced wood as the major materials for bridge construction, wood is still widely used for short and medium span bridges. Of the bridges in the United States with spans longer than 20 feet, approximately 12 percent are made of timber. In the USDA Forest Service alone, approximately 7,500 timber bridges are in use, and more are built each year [1].

The advantages of timber bridges include lightweight, high span-to-weight ratio, durability, capability of supporting short-term overloads without adverse effects, good fire resistance qualities, competitiveness in cost terms with other materials in small bridge construction, and immunity to deteriorating effects of de-icing agents. Timber bridges can be constructed in any weather condition, and do not require special installation equipment. They also present a natural and aesthetically pleasing appearance.

Glulam, which is the most widely used modern timber bridge material, is manufactured by bonding sawn lumber laminations together with waterproof structural adhesives. Thus, glulam members can be manufactured in a wide range of shapes with unlimited length, width and depth. Glulam provides higher design strengths than sawn lumber and permits manufacture of large wood structural elements from smaller lumber sizes. Technological advances in laminating have further increased the suitability and performance of wood for bridge applications.

1.2 Background

Over the past few years, research has been conducted on longitudinal timber deck bridges and girder bridges at Iowa State University (ISU). In 1985, ISU conducted an analytical study to develop design criteria for the live load distribution of longitudinal deck bridges. Additional experimental and analytical work was carried out to investigate the behavior of longitudinal deck bridges by Funke [2], Bisat [3], and Hajdu [4] at ISU. Bhari [5] did a sensitivity study on the connection between girders and the deck in girder bridges by using various analytical models. The dynamic behavior of timber girder bridges and stress laminated timber bridges was studied in detail by Dlabola [6] and Johnson [7], respectively.

A number of analytical models have been developed in past work to successfully analyze the behavior of timber bridges. With the computer age, finite element analysis has become extensively popular in the analysis of complex structures. In previous analytical modeling of longitudinal deck bridges and girder bridges, Funke [2], Bisat [3], Hajdu [4], Bhari [5], Dlabola [6], and Johnson [7] have used the finite element method.

1.3 Properties of wood

Wood is an orthotropic material with unique and independent properties in different directions. Because of the orientation of the wood fibers, and the manner in which trees increase in diameter as they grow, properties vary along three mutually perpendicular axes: longitudinal, radial and tangential (Fig. 1.1). Most wood properties for structural applications are given only for directions parallel (longitudinal) and perpendicular to the grain (radial and tangential) since the differences in wood properties between the radial and tangential directions is minor compared to their mutual differences in the longitudinal direction.
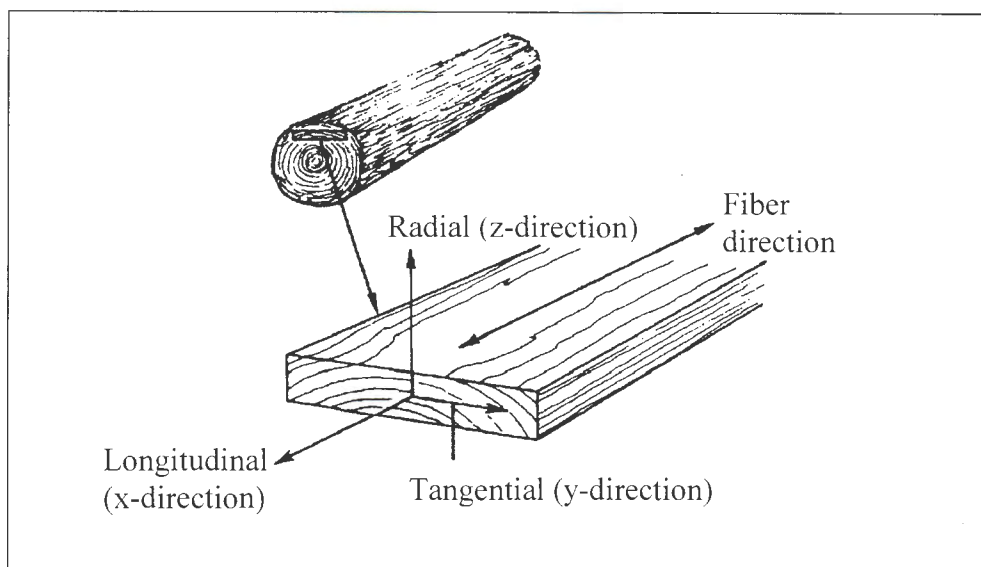
Fig. 1.1 The three principal axes of wood [1].

The ANSYS finite element software [8] denotes these material properties by associating them with the corresponding material axes (Fig. 1.1), as shown below:

$E_{x,y,z}$ – Young's modulus in the longitudinal, tangential and radial directions, respectively.
$G_{xy,yz,zx}$ – Shear modulus in the x-y, y-z and z-x planes, respectively.
$\nu_{xy,yz,zx}$ – Major Poisson's ratio in the x-y, y-z, and z-x planes, respectively.

The ANSYS software allows the user to input E_x, E_y, and G_{xy}. E_z defaults to E_y, and G_{yz}, and G_{zx} default to G_{xy}. The user is also given an option to enter Poisson's ratio, ν_{xy}, which defaults to the value of zero. The ANSYS software relates the state of stress and strain in a body by the elasticity matrix, [D]. Fatal errors occur in the program if the inverse of the elasticity matrix, $[D]^{-1}$, is not positive definite. The $[D]^{-1}$ matrix is also presumed to be symmetric. The use of Poisson's ratios for orthotropic materials causes confusion, so care should be taken in their use. To assure that the $[D]^{-1}$ matrix is positive definite and symmetric, the following relationship must be satisfied:

$$\nu_{ij} = \nu_{ji}\, E_i\, /\, E_j$$

where   i,j = x,y,z , and i ≠ j.

G_{xy} (default) $= E_x E_y / (E_x - (1+2\nu_{xy})E_y)$

$$[D]^{-1} = \begin{pmatrix} 1/E_x & -\nu_{xy}/E_x & -\nu_{xy}/E_x & 0 & 0 & 0 \\ -\nu_{yx}/E_y & 1/E_y & -\nu_{yz}/E_y & 0 & 0 & 0 \\ -\nu_{zx}/E_z & -\nu_{xy}/E_z & 1/E_z & 0 & 0 & 0 \\ 0 & 0 & 0 & 1/G_{xy} & 0 & 0 \\ 0 & 0 & 0 & 0 & 1/G_{yz} & 0 \\ 0 & 0 & 0 & 0 & 0 & 1/G_{xz} \end{pmatrix}$$

1.4 Objective and scope of report

Iowa State University (ISU) and the Forest Products Lab (FPL) at Madison, Wisconsin have done extensive load testing of timber bridges around the country to study the behavior of timber bridges and improve upon the existing design methodologies for various types of timber bridges. The author's work has been to develop an effective analysis tool to aid ISU and FPL with their bridge evaluation. Hence, the objective of this study is three-fold:

1. To make efficient finite element models of the longitudinal glulam deck bridge and the glulam girder bridge that would aid in accurately predicting the behavior of these bridges. The finite element models must include all bridge components that contribute to the structural behavior of the bridge.

2. To develop user friendly pre- and post-processors in ANSYS [8] that would not require an in-depth knowledge of the finite element method and need only minimum

user input. The pre-processors model and analyze the bridges while the post-processors output the deflections and stresses at key locations on the bridge.

3. To perform a parametric study to investigate the sensitivity of analytical results to varying material properties, end restraints, edge stiffening and the manner in which loads are applied to the model.

The above objectives were accomplished by analyzing two right-angled glued laminated deck bridges, three right-angled girder bridges and one skew girder bridge. The analytical results were compared to available experimental data and the models were validated.

1.5 Software Selection

The analysis of longitudinal deck bridges and girder bridges was accomplished by using ANSYS 5.5 finite element software for structural analysis [8]. ANSYS was considered appropriate because it is a powerful, general-purpose software that is capable of analyzing complex structures. The software is well established, well calibrated and has an excellent graphical user interface.

2. LONGITUDINAL GLULAM TIMBER DECK BRIDGES

2.1 Description of deck bridges

Longitudinal deck bridges consist of a glulam timber deck placed over two or more substructure supports (Fig. 2.1). The lumber laminations are placed parallel to traffic, and loads are applied parallel to the wide face of the laminations. Transverse stiffener beams are connected to the deck underside to distribute loads laterally across the bridge width. Longitudinal deck bridges provide a low profile that makes them especially suitable for short-span applications where clearance below the structure is limited.

Longitudinal deck bridges consist of a series of glulam panels placed edge to edge across the deck width. They are practical for clear spans up to approximately 35 feet and are equally adaptable to single-lane and multiple-lane crossings. The panels are usually not interconnected with dowels or fasteners, but are provided with transverse stiffener beams below the deck. These stiffener beams are bolted to the panels directly (through-bolts) or with brackets, and their main function is to provide lateral or transverse continuity to the system. They are also frequently used as a point of attachment for railing systems. The configuration of a typical longitudinal deck bridge is shown in Fig. 2.2.

The deck panels are usually 42 to 54 inches wide in increments equal to net lamination thickness (1.5 inches for western species and 1.375 inches for Southern Pine). They can be manufactured in any length subject to local pressure treating and transportation restrictions. The deck thickness is usually between 5 inches to 10.75 inches.

(a) Side view.

(b) Bottom view showing stiffener beams connected to the deck with thru-bolts.

Fig. 2.1 Generic photo of a longitudinal glulam timber deck bridge.

(a) Top view.

(b) Side view.

(c) Cross-section view.

Fig. 2.2 Configuration of a typical longitudinal deck bridge.

(d) Connections between deck and stiffener.
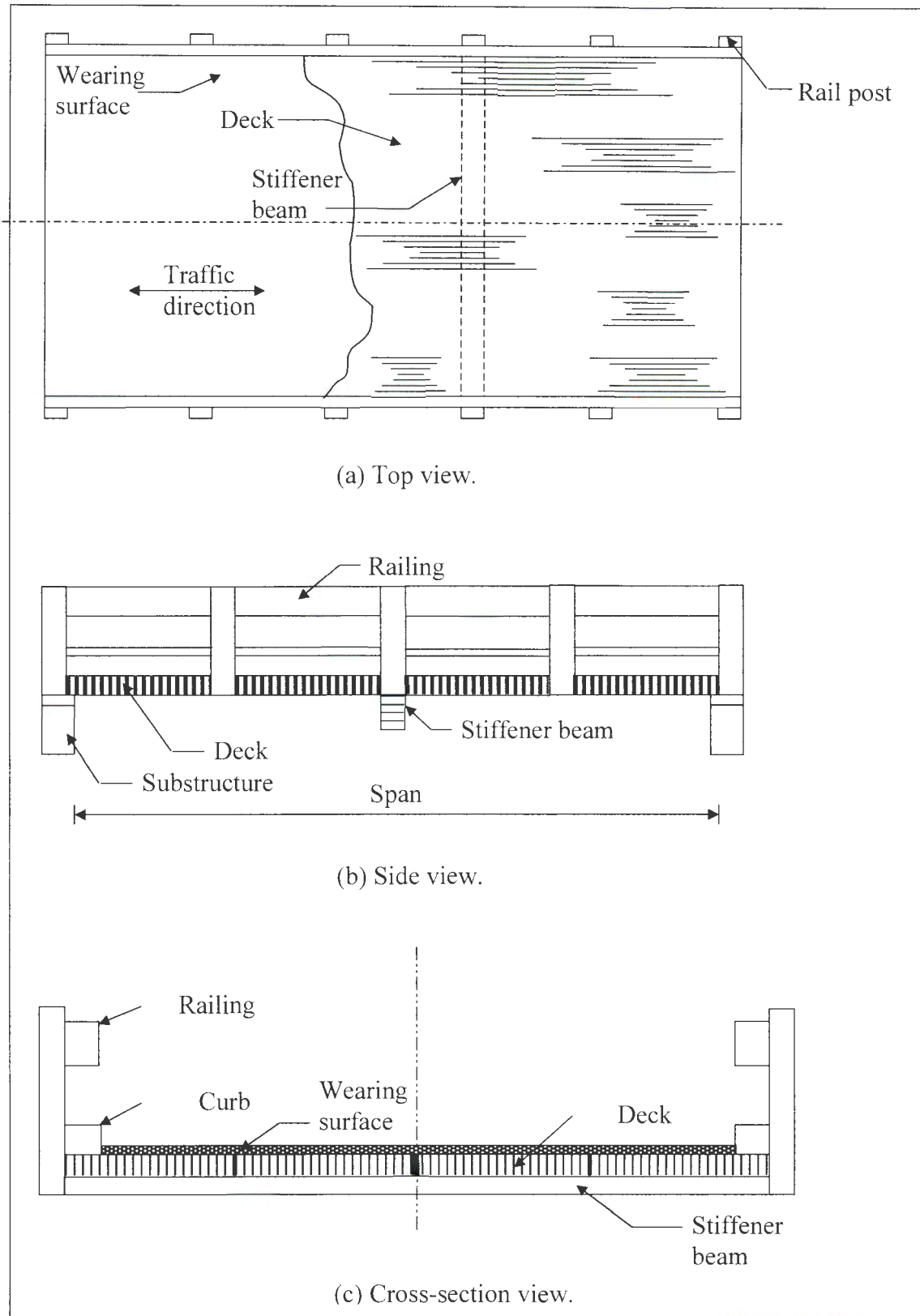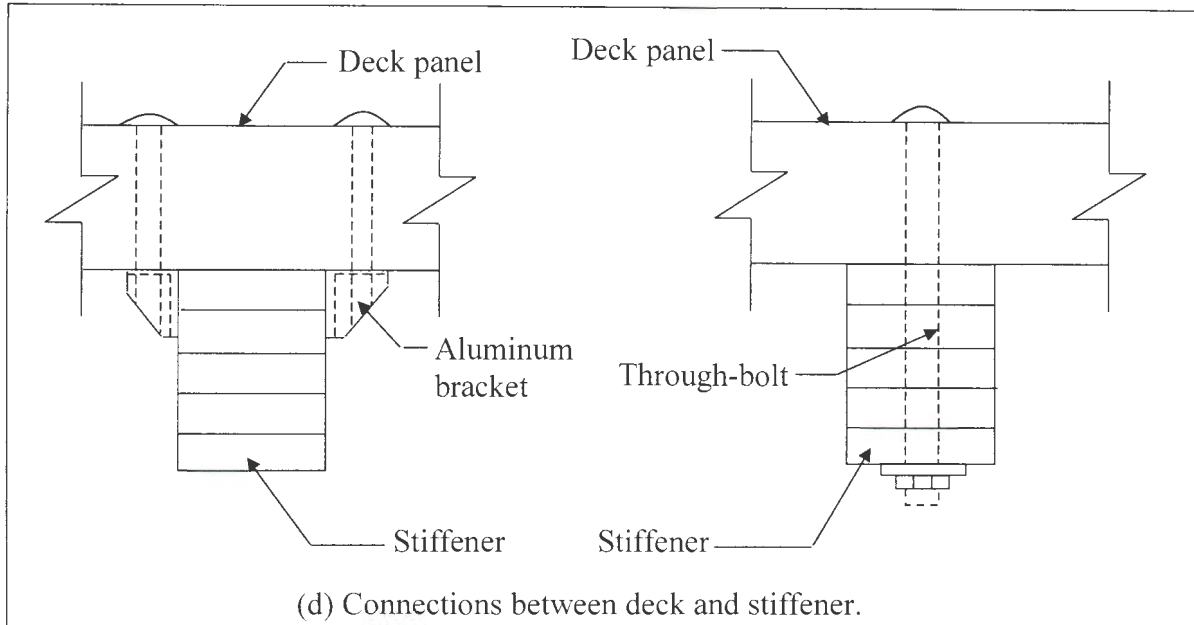
Fig. 2.2 Continued.

2.2 Design procedures for deck bridges

Deck panels for longitudinal glulam bridges are designed as individual glulam beams of rectangular cross-section. Distribution factors are used to distribute the vehicle wheel line load to each panel. The bending, deflection, shear, and reactions distributed to each panel are assumed to be resisted by the entire panel cross-section. The design procedures for longitudinal deck bridges are explained in greater detail below. These procedures are valid for panels that are 3.5 – 4.5 feet wide and are provided with transverse stiffener beams [1].

Initially, the basic geometric requirements for the deck are defined. The effective deck span, 'L', is defined as the distance measured center-to-center of bearings. The deck width is the roadway width plus any additional width required for curb and railing systems. The design loads and load distribution criteria are also determined beforehand. The deck thickness and width must be estimated for initial calculations. Approximate deck thickness

for certain maximum deck spans may be obtained from [1]. Based on the estimated panel

dimensions, properties for the panel cross-section are computed as follows:

$$\text{Panel area (in}^2) = A = w_p t$$
$$\text{Section modulus of panel (in}^3) = S_y = w_p t^2/6$$
$$\text{Moment of inertia of panel (in}^4) = I_y = w_p t^3/12$$

where w_p = panel width (in), and
t = panel thickness (in).

When railings and curbs are supported by transverse stiffener beams, the dead load is

normally assumed to be equally distributed to all panels. When railings and curbs are

attached to the outside panel, their dead load is included with the dead load of the panel.

Longitudinal glulam panels are designed as individual members to resist applied loads. In the

direction of the deck span, no longitudinal distribution of wheel loads is assumed, and wheel

loads act as concentrated loads. The portion of the wheel line laterally distributed to each

panel is based on the Wheel Load Fraction (*WLF*).

$$WLF = w_p / (4.25 + L/28) \text{ OR } w_p / 5.50, \text{ whichever is greater (one traffic lane)}$$
$$= w_p / (3.75 + L/28) \text{ OR } w_p / 5.00, \text{ whichever is greater (two or more traffic lanes)}$$

where w_p = panel width in feet, and
L = deck span in feet measured center-to-center of bearings.

Based on the magnitude of the deck bending stress, 'f_b', a panel combination symbol

with the required bending capacity is selected from Table 2 of AITC 117-Design [9]. If the

applied bending stress, 'f_b', is greater than the allowable bending stress, 'F_b'', the deck is

insufficient in bending and the deck thickness or grade must be increased. If deck thickness

or width is changed, the design procedures must be repeated. If $f_b \le F_b'$, the initial deck

thickness and combination symbol are satisfactory in bending.

$$f_b = (M_{DL} + M_{LL})/ S_y$$
$$M_{DL} = w_{DL} L^2 / 8$$

$$M_{LL} = M_{WL} \ (WLF)$$
$$F_b' = F_{by} \ C_F \ C_M$$

where M_{DL} = dead load moment of one panel,
                M_{LL} = live load moment applied to one panel,
                w_{DL} = panel dead load,
                M_{WL} = maximum moment produced by one wheel line of the design vehicle,
                F_{by} = tabulated bending stress from [9] for particular combination symbol,
                C_F = size factor for panels less than 12 inches thick [1], and
                C_M = wet-use factor for glulam = 0.80 [1].

Live load deflection is resisted by the full moment of inertia, 'I_y', of the panel section. The deflection applied to each panel is the maximum deflection produced by one wheel line of the design vehicle times the 'WLF'. The deck live load deflection is computed by standard methods of elastic analysis, with the glulam modulus of elasticity, 'E', adjusted for wet-use conditions. Requirements for live load deflection are not included in AASHTO specifications, and the acceptable deflection limit is left to designer judgment [1]. Since continuity from panel to panel is provided only at stiffener beam locations, relative panel displacements do occur at locations between these beams. Research at ISU [10] indicates that the inter-panel displacement will not exceed approximately 0.1 inch in most applications. A further reduction is desirable to reduce the potential for asphalt cracks at the panel joints, or when the bridge includes a pedestrian walkway.

Horizontal shear is normally not a controlling factor in longitudinal deck design because of the relatively large panel area. Dead load vertical shear is computed at a distance from the support equal to the deck thickness, 't', neglecting the loads within the distance 't' from the supports. Live load vertical shear is based on the maximum vertical shear occurring at a distance from the support equal to three times the deck thickness ($3t$), or the span quarter point ($L/4$), whichever is less. Horizontal shear stress is assumed to be resisted by the total

area of the panel cross-section, '*A*'. Applied stress, '*f$_v$*', must not be greater than the allowable shear stress for deck combination symbol, '*F$_v$*'':

$$V_{DL} = w_{DL} (L/2 - t)$$
$$V_{LL} = V_{WL} (WLF)$$
$$F_v = 1.5 \ V/A \leq F_v' = F_{vy} \ C_M$$

where $V = V_{DL} + V_{LL}$,
             F_{vy} = tabulated shear stress from [9] for particular combination symbol, and
             C_M = wet-use factor for shear = 0.875 [1].

In practice, stiffener beams are used for guardrail post attachment, and therefore, stiffener spacing, strength, and connections may be dictated by more restrictive railing requirements. AASHTO specifications (AASHTO 3.25.3.4) require that a stiffener beam be placed at midspan for all deck spans, and at intermediate spacings not to exceed 10 feet. The American Institute of Timber Construction (AITC), however, recommends an intermediate stiffener beam spacing of 8 feet [1]. Stiffener design consists of sizing the beam so that the stiffness factor, '*EI*', of the member is not less than 80,000 kips-square inch (k-in^2); however, this is an approximate value that should not be significantly exceeded. Load distribution between panels is more effectively improved by decreasing stiffener beam spacing, rather than by increasing the beam size substantially above the required minimum [1].

Connections between the stiffener beam and the deck panels are placed approximately 6 inches from each panel edge. The type of connection depends on the stiffener-beam material and configuration. Through-bolting is used for glulam beams and steel channels. Deck brackets or steel plates are also used for glulam beams, and C-clips are used for steel I-beams. Research at ISU [10] indicates that the through-bolt type of connection provides more favorable load distribution in the panels and reduces the potential for localized stress conditions in the region of the connection to the stiffener beams. They are

also more effective in reducing inter-panel displacements that occur between stiffener beam locations.

For longitudinal deck bridges, the required bearing length is normally controlled by considerations for bearing configuration, rather than stress in compression perpendicular to the grain. A bearing length of 10 to 12 inches is usually sufficient for stability and deck attachment. The bearing attachments are normally made through the deck to the supporting cap or sill, or from the deck underside. For short-span crossings, a side attachment using steel angles may also be feasible.

Based on the bearing configuration, the dead load reactions are computed by conventional methods using the unit dead load of the panel. Live load reactions for single and multiple lane bridges are based on 'WLF' from AASHTO 3.25.3.2. The live load reaction distributed to each panel is the maximum reaction of the design vehicle times the 'WLF'. Applied stress in compression perpendicular to the grain reactions, '$f_{c\perp}$', must not be greater than the allowable stress in compression perpendicular to the grain for the panel combination symbol, '$F_{c\perp}$'':

$$R_{LL} = R_{WL} \, (WLF)$$
$$WLF = w_p \, / \, 4, \text{ but not less than } 1.0$$
$$f_{c\perp} = (R_{DL} + R_{LL}) \, / \, (w_p \, l_b) \leq F_{c\perp}' = F_{c\perp} \, C_M$$

where R_{LL} = live load reaction on one panel,
           R_{WL} = maximum reaction produced by one wheel line of design vehicle,
           R_{DL} = dead load reaction on one panel,
           l_b = length of bearing,
           $F_{c\perp}$ = tabulated stress in compression perpendicular to the grain from [9] for
                    particular combination symbol, and
           C_M = wet-use factor for bearing = 0.53 [1].

2.3 Analytical model of bridge

2.3.1 Modeling of deck panels

The deck panels are laid out longitudinally between the supports. The deck panels are non-interconnected and the assumption made in this regard is that the asphalt-wearing surface and friction between the adjacent panels contribute insignificantly towards continuity and load distribution between the panels.

The four-node shell element (SHELL63) [11] was chosen to model the deck panel. The element chosen has six degrees of freedom at each node: translation in the nodal x, y, and z directions and rotations about the nodal x, y, and z-axes. This element can be used to model the orthotropic properties of wood and is defined by thickness, longitudinal and transverse moduli of elasticity, shear modulus and major or minor Poisson's ratio. The element permits both in-plane and lateral loads and has both bending and membrane capabilities.

The longitudinal modulus of elasticity (parallel to the grain of fiber) of the panels is substantially higher than the transverse modulus of elasticity (perpendicular to the grain of fiber) and the modulus of elasticity tangential to the grain of fiber. Fig. 2.3 shows the SHELL63 element used to model the deck panels.

2.3.2 Modeling the stiffener beams

The stiffener beams provide lateral continuity to the deck panels. In some cases, they also provide for attachment of the railing system to the bridge. The two-node 3-D beam element (BEAM4) [11] was chosen to model the stiffener beam. This element is a uniaxial freedom at each node: translation in the nodal x, y, and z directions, and rotations about the

15
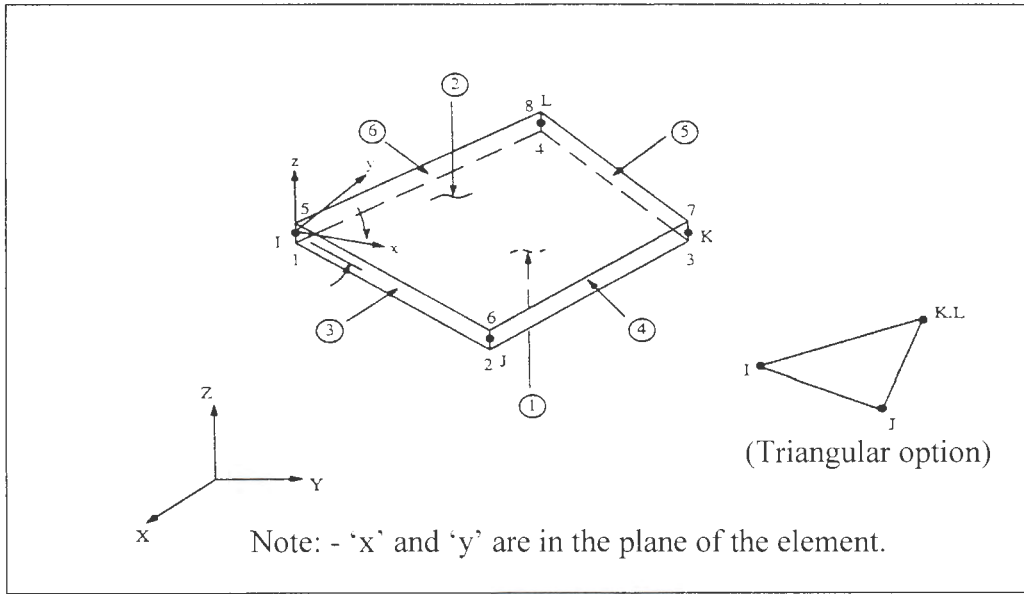
Fig. 2.3 SHELL63 element used to model the deck panels [11].

element with tension, compression, torsion, and bending capabilities, and has six degrees of

nodal x, y, and z-axes. The stiffener beams were assumed to be isotropic in bending. The

beam element is defined by thickness, width, area, inertia about the y- and z-axes,

longitudinal modulus of elasticity, shear modulus, and major or minor Poisson's ratio. Fig.

2.4 shows the BEAM4 element used to model the stiffener beams.
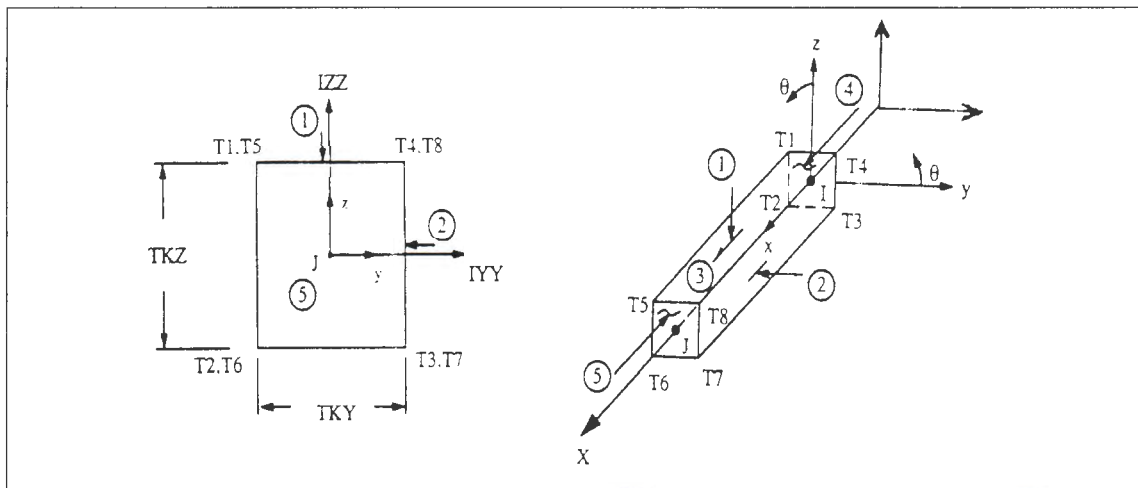
Fig. 2.4 BEAM4 element used to model the stiffener beam and curbs [11].

2.3.3 Modeling the connections

The longitudinal glulam timber deck is connected to the transverse stiffener beams with through-bolt or aluminum bracket connections. Research at ISU [10] has shown that the interpanel displacements are lesser when the stiffener beams are connected to the deck with through-bolts. In the model, both these types of connections were treated as rigid connections. These connections were modeled using the 3-D beam element, BEAM4 (Fig. 2.4), with flexural and axial stiffness equal to 100,000 kips per inch (very high value).

2.3.4 Modeling the curbs

The curbs were modeled using 3-D beam (BEAM4) elements. Fig. 2.4 illustrates the orientations of the three axes. A consistent tangent stiffness matrix option is available for use in large deflection (finite rotation) analysis. Beam elements, with dimensions and longitudinal modulus of elasticity of the curbs, ran along either edge of the span of the bridge and were connected to the deck by rigid links. In modeling these rigid links, the deck nodes were assigned to be the master nodes and the curb nodes were assigned to be the slave nodes. The preprocessor gives the user the option to include curbs in the model. This option becomes very useful when a parametric study of the behavior of curbs on the bridge needs to be done.

It should be noted that the dead load or the permanent weight of all the structural and non-structural components of the bridges, including the roadway, sidewalks, railing, and wearing surface, were not included in the load. Their contribution to the behavior of the deck bridge was assumed to be insignificant.

2.3.5 Modeling the loads and abutment supports

The bridge was assumed to be simply supported. This assumption was considered to be conservative because in reality, there may be some rotational fixity near the abutments. The live load applied was truck wheel loads. The wheel contact areas were assumed to be small relative to the bridge and hence, were applied as concentrated point loads. The dead loads of the deck, the wearing surface, curbs, and the railings were neglected. Field data were measured due to the effects of live loads only. The ANSYS software requires concentrated loads at nodes. The concentrated wheel loads were distributed to the nodes in the form of energy equivalent loads, since very rarely did the location of a wheel load correspond to the location of the panel node.

In finite element analysis, nodal loads for an element are calculated using the energy approach that utilizes the same shape functions to develop the stiffness matrix of the particular element. If $\{N\}$ is the vector containing the shape functions of an element, and 'P' is the applied concentrated load on the element, the nodal load vector $\{f\}$ is calculated as:

$$\{f\} = \{N\}^T P$$

Since shape functions for the SHELL63 element were not explicitly stated in ANSYS [11], approximate shape functions by Desai [12] were used in the preprocessor to calculate the energy equivalent loads for a rectangular plate element. This approach was first used by Jain [13] in his preprocessor for stress laminated timber bridges. Desai developed the shape functions for a rectangular plate element with four nodes having three degrees of freedom at each node: translation in the z-direction and rotations about the x and y-axes. In the formulation discussed below, node '1' refers to the bottom left node of the rectangular element, node '2' refers to the bottom right node, node '3' refers to the upper right node, and

node '4' refers to the upper left node of the rectangular element. Refer to Fig. 2.5 for directions of the axes and the different degrees of freedom at each node. The rectangular element discussed below, is assumed to have a length of 'a' in the x-direction and a width of 'b' in the y-direction and the out-of-plane concentrated load 'P' is assumed to be at a distance 'x' in the x-direction and 'y' in the y-direction, on the element, measured from the bottom left corner of the element. The equivalent nodal loads are given below:

For vertical loads, F_z:	For moments, M_x:	For moments, M_y:
$N_z(1) = nx1*ny1$	$N_x(1) = nx3*ny1$	$N_y(1) = nx1*ny3$
$N_z(2) = nx2*ny1$	$N_x(2) = nx4*ny1$	$N_y(2) = nx2*ny3$
$N_z(3) = nx2*ny2$	$N_x(3) = nx4*ny2$	$N_y(3) = nx2*ny4$
$N_z(4) = nx1*ny2$	$N_x(4) = nx3*ny2$	$N_y(4) = nx1*ny4$

where
$$nx1 = 1 - 3s^2 + 2s^3 \qquad ny1 = 1 - 3t^2 + 2t^3,$$
$$nx2 = s^2(3 - 2s) \qquad ny2 = t^2(3 - 2t),$$
$$nx3 = a\,s\,(s-1)^2 \qquad nx3 = b\,t\,(t-1)^2,$$
$$nx4 = a\,s^2(s-1) \qquad ny4 = b\,t^2(t-1),$$
$$s = x/a \qquad t = y/b,$$

$N_z(i)$ = interpolation function for force in z-direction of element at node 'i',
$N_x(i)$ = interpolation function for moment about y-axis of element at node 'i',
$N_y(i)$ = interpolation function for moment about x-axis of element at node 'i'.
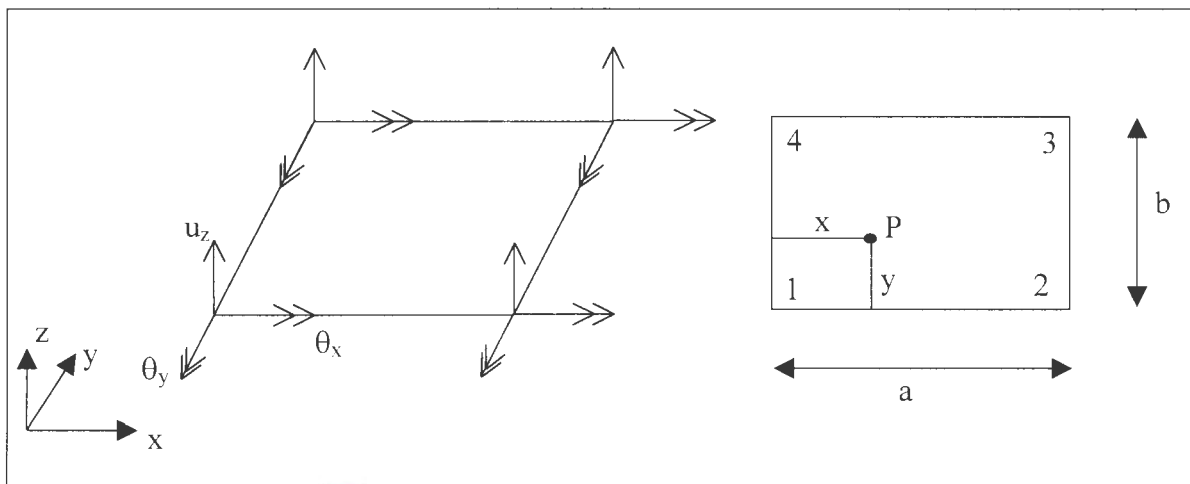
Fig. 2.5 Sign convention used by Desai [12] for his interpolation functions.

2.4 Mesh size determination

A mesh sensitivity study was done on the analytical model of the bridge to determine the adequate size of the element to be used. The element size should be such that accurate results are obtained in minimum computer time. This element size depends on factors such as structure geometry, loading pattern, and boundary conditions and hence varies for each bridge; however, since the deck panel has a limited width range of 42 inches to 54 inches, it was decided to use a pre-determined element size in the preprocessor. This element size was determined from a mesh sensitivity study on a fictitious simply supported bridge (Fig. 2.6) with span-length of 26 feet.

The bridge consisted of six deck panels, each panel being 4 feet wide, and 10.75 inches thick. Three stiffener beams of cross-section 6.75 inches by 4.5 inches supported the deck panels underneath. The deck had a longitudinal modulus of elasticity of 2100 ksi, a transverse modulus of elasticity of 240 ksi, and a shear modulus of 106 ksi. The stiffener beams had a modulus of elasticity of 1625 ksi, and a shear modulus of 80 ksi. The bridge had curbs of size 8 inches by 8 inches on each edge. Two concentrated loads of 16 kips each were applied on the transverse centerline of the bridge at 108 inches from each edge of the bridge. The above values were chosen so that they closely represented an actual longitudinal glulam deck bridge. The aspect ratio was kept as close to unity as possible.

The results of the analysis are shown in Fig. 2.7 and Table 2.1. It can be observed that when the element size was 18 inches by 12 inches, midspan panel edge deflections and maximum stiffener beam bending stresses started converging. As the field bridges may differ in dimension, the element size calculated by the program may differ slightly from the above mentioned element size.
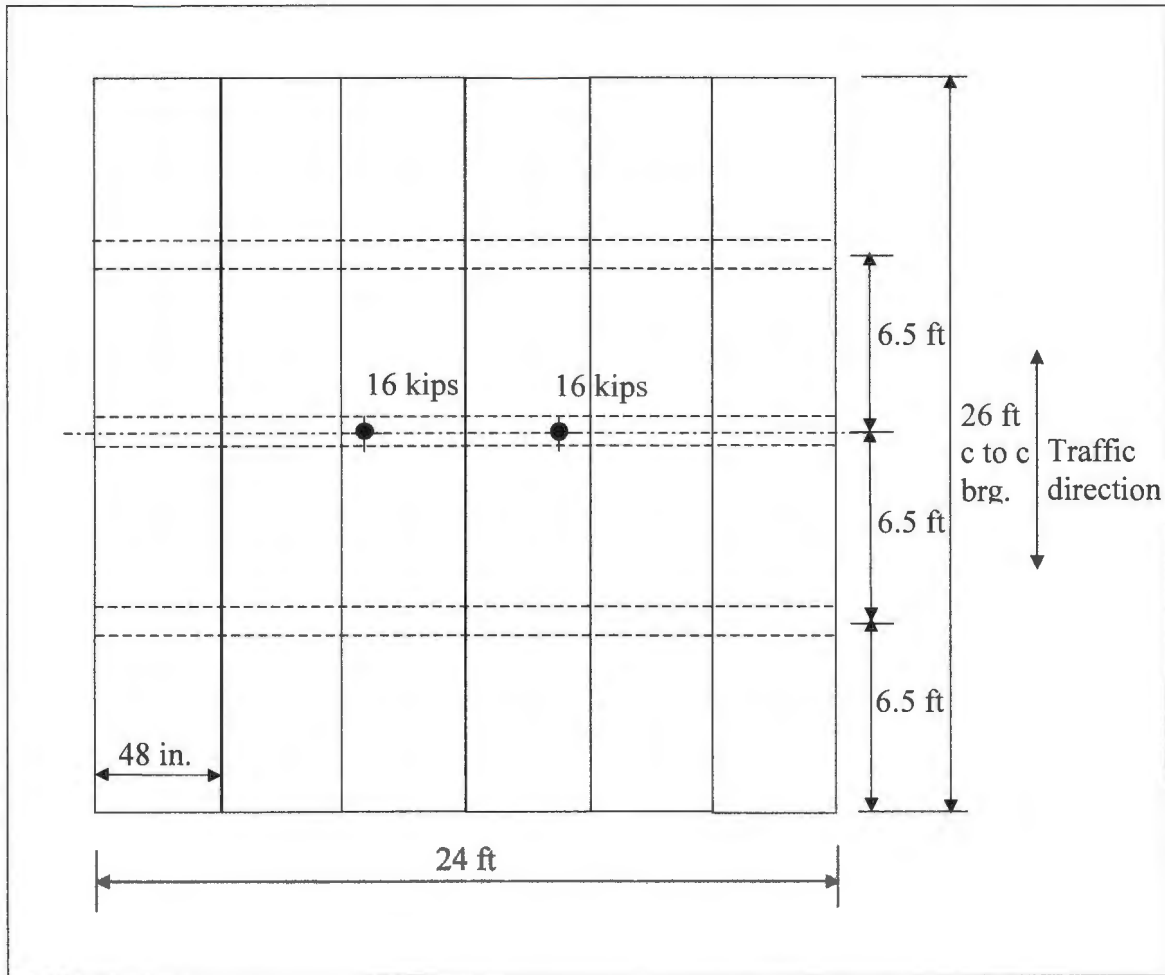
Fig. 2.6 Plan layout of bridge used for mesh sensitivity analysis.
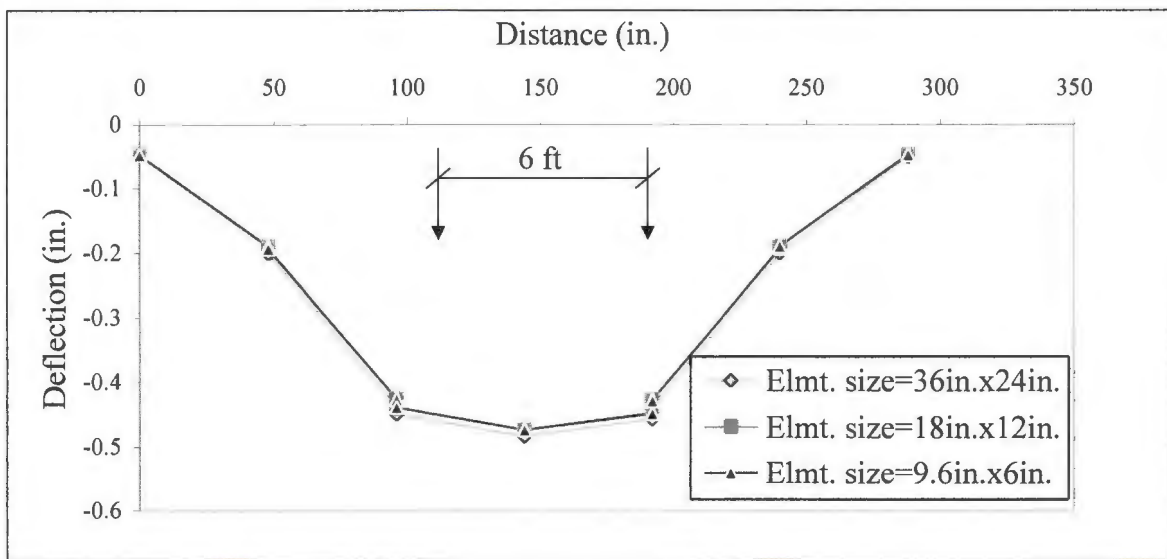
Fig. 2.7 Panel deflections at midspan for mesh sensitivity analysis.

Table 2.1 Maximum bending stresses for panel and stiffener beam at midspan.

Element size	Maximum panel longitudinal stress (ksi)	Maximum stiffener beam bending stress (ksi)
36in.x24in.	0.65	0.95
18in.x12in.	**0.69**	**0.98**
9.6in.x6in.	0.69	0.98

2.5 Preprocessor and postprocessor for longitudinal glulam deck bridges

These programs were developed using ANSYS Parametric Design Language (APDL) that is available inside the ANSYS software. The purpose of these programs was to reduce user input and provide required results in a more presentable format. The inputs required for the programs are: 1) basic dimensions of the bridge, 2) material properties of the deck, stiffener beams, and curbs (if provided), and 3) position and magnitude of the loads.

These programs store the required output in a tabular format in a file called 'Results.dat', that is created in the home directory of the user. This output can then be imported into an Excel spreadsheet and the user can obtain a graphical representation of the results. The programs are capable of analyzing only single spans and the results obtained are along the midspan of the bridge. The output file contains midspan panel edge deflections, panel stresses (longitudinal, transverse, and shear) for user-selected panels, and midspan stiffener beam bending stresses. If a stiffener beam does not exist at midspan, then the bending stresses for the stiffener beam closest to midspan is listed in the output file. A listing of the program with comments is provided in Appendix A. A complete 'User Manual' to the program with its limitations, and an example is provided in Appendix C. Fig. 2.8 shows the analytical model of a typical glulam deck bridge.

(a) Model of deck panel.

Deck panel

Stiffener
beam

(b) Model of stiffener beam.

(c) Model of deck bridge.
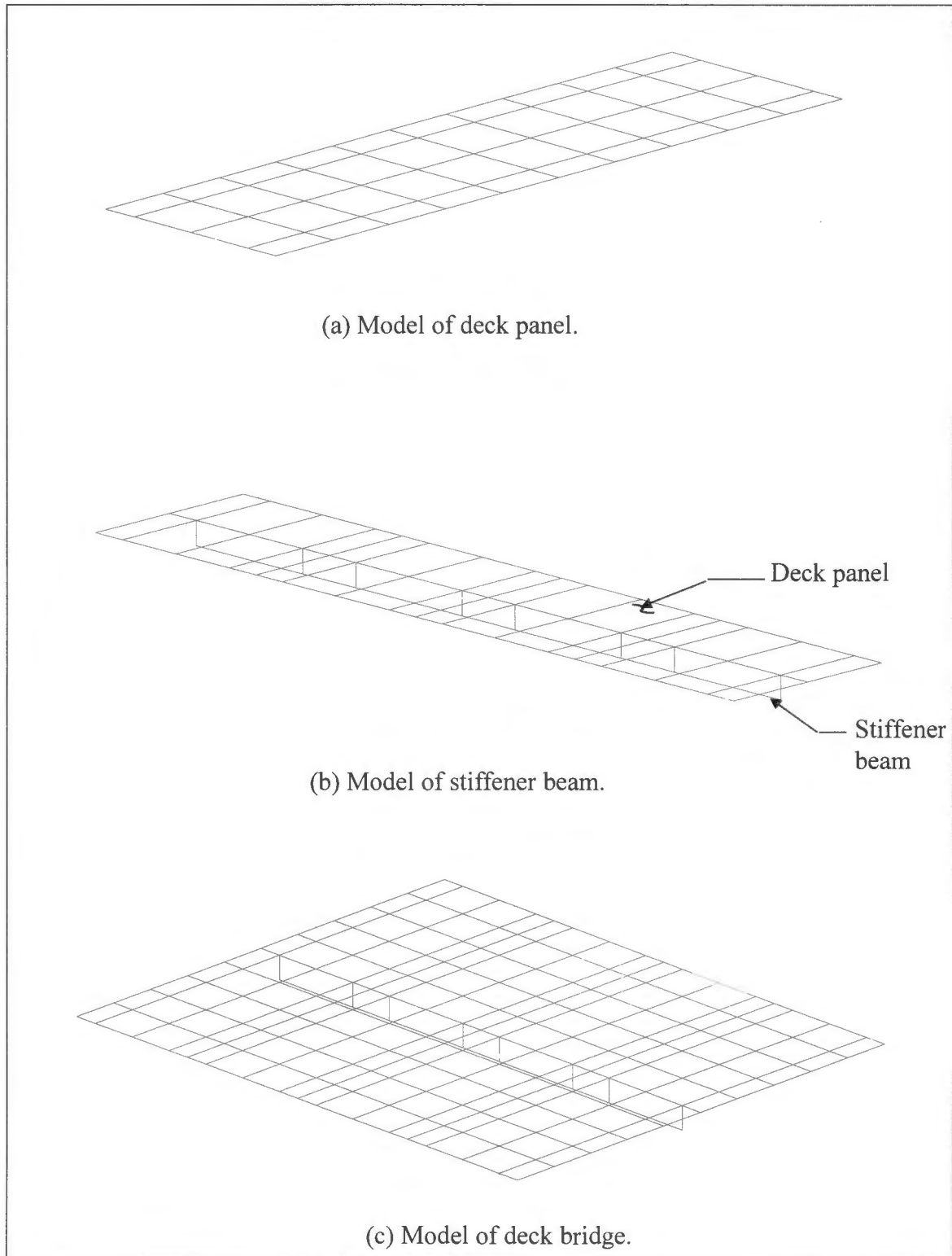
Fig. 2.8 Analytical model of a typical deck bridge with four panels and one stiffener beam.

3. COMPARISON OF EXPERIMENTAL AND ANALYTICAL DATA FOR LONGITUDINAL DECK BRIDGES

3.1 General

Two case studies were analyzed with the model developed. The case studies were the Angelica Bridge in New York (NY), and the Bolivar Bridge in NY. These are both right-angled and short-span bridges. A parametric study was also performed for each bridge to determine the sensitivity of the results. This study was done by varying different input parameters such as material properties of the deck and stiffener beams, end restraint, stiffener beam cross-section dimension, and curb height. The above were first performed on the Angelica Bridge. Based on this study, the parametric study on the Bolivar Bridge was done only for parameters that had a significant effect on the results.

3.2 The Angelica Bridge, NY

3.2.1 Description

The Angelica Bridge (Fig. 3.1) is a two-lane, glulam deck bridge with eight panels, and four stiffener beams running transverse to the longitudinally placed deck panels. The stiffener beams are connected to the deck panels by through-bolt connections. The bridge deck is made of Douglas Fir timber. The deck panels are about 4.5 feet wide, 30 feet long, and 14.5 inches thick and are placed longitudinally between supports. These deck panels are not interconnected. The stiffener beams are connected to the deck panels by through-bolts at 6 inches in from each edge of the panel. The bridge measured about 30 feet in span,

(a) End view of the Angelica Bridge showing the railing unit.

(b) Elevation view of the Angelica Bridge.

Fig. 3.1 The Angelica Bridge in NY.

measured center-to-center of bearings. The stiffener beams have a nominal width of 6.75 inches, and a nominal thickness of 4.25 inches. The bridge has deep curbs on its edges with a cross-section dimension of 8 inches by 21.5 inches. The material properties for the analytical model were obtained from the Douglas Fir Handbook [14] for the particular grade of lamination used on the bridge. The deck panels have a longitudinal and transverse modulus of elasticity of 1700 kips-per-square inch (ksi) and 240 ksi, respectively. The shear modulus of the deck panels is about 100 ksi. The stiffener beams have a longitudinal modulus of elasticity of 1625 ksi. The shear modulus of the stiffener beams is about 80 ksi. Refer to Fig. 3.2 for the design configuration of the bridge.
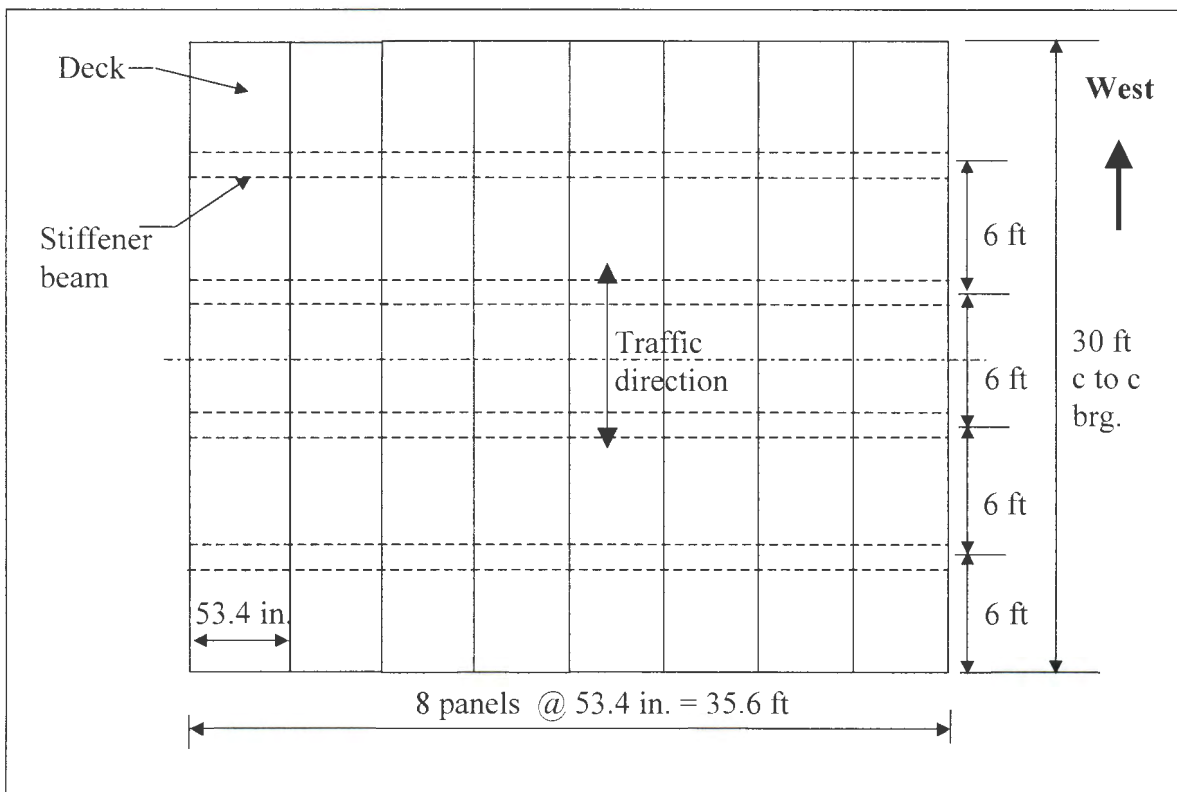
Fig. 3.2 Plan layout of the Angelica Bridge.

3.2.2 Loading

Six load tests were done on the bridge. The load test vehicles consisted of two fully loaded three-axle dump trucks with gross vehicle weights of 70,320 pounds (Truck 18) and 75,940 pounds (Truck 12). The rear axles on Truck 18 weighed about 52,900 pounds while the rear axles on Truck 12 weighed about 55,540 pounds. The rear axles weighed about 75% of the respective gross vehicle weights. The vehicles were positioned longitudinally on the bridge so that the two rear axles were centered on the midspan of the bridge. Since this is a short-span bridge, the front axle of the trucks was outside the bridge for all load cases. The transverse vehicle track width, measured center-to-center of the rear tires, was 6 feet. For Load Cases 1 and 4, Truck 12 was placed on the east lane of the bridge, 24 inches and 104 inches away from the longitudinal centerline of the bridge, respectively. For Load Cases 2 and 5, Truck 18 was placed on the west lane of the bridge, 24 inches and 104 inches away from the longitudinal centerline of the bridge, respectively. Load Case 3 was a combination of Load Cases 1 and 2, while Load Case 6 was a combination of Load Cases 4 and 5. Fig. 3.3 shows the configuration of the trucks and the load positions on the bridge.

3.2.3 Parametric study

A parametric study on the Angelica Bridge was done for Load Cases 1 and 3. The input data described in Section 3.2.1 was used for the initial run on the program. The material properties of the deck were then varied and a comparative plot for panel edge deflections at midspan of bridge was made. Comparison was also done with respect to the experimental data from field tests. Similar deflection comparisons were made by individually changing the curb height, material properties of the stiffener beam, stiffener beam cross-section dimension, and end
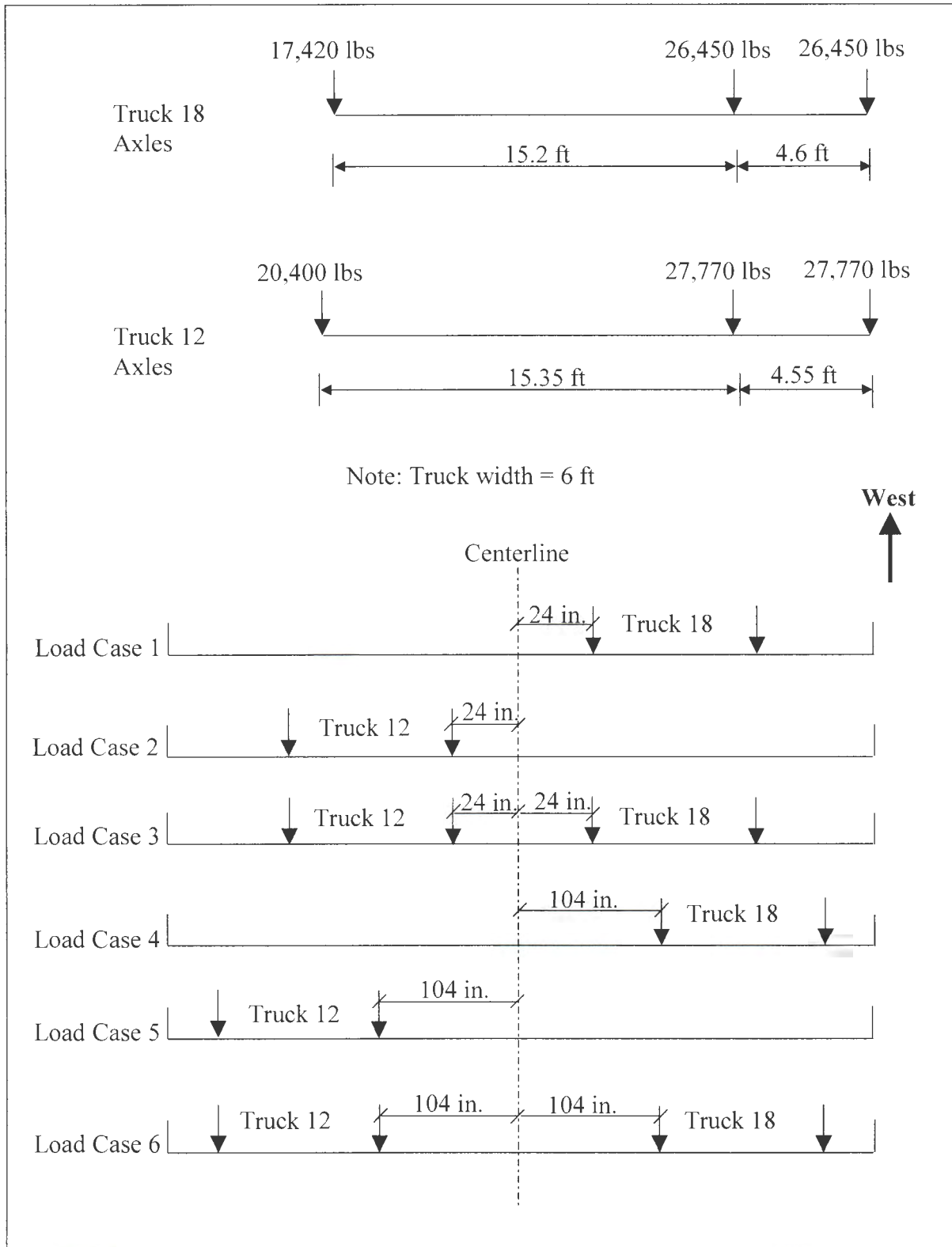
Fig. 3.3 Configuration of trucks and load positions for various load cases.

restraint. The analytical model used for the study is shown in Fig. 3.4. Figs. 3.5 to 3.10 show the comparative plots for the full parametric study.

A comparative study of experimental values and analytical results for midspan panel edge deflections was also done for all the other load cases. The deflection plots for this comparison are shown in Figs. 3.11 to 3.14.

3.2.4 Comparison of maximum design stress and maximum stress from analysis

The maximum design stress for the panels on the Angelica Bridge was computed using the procedure described in Section 2.2. The maximum longitudinal panel stress at midspan obtained using the program was 880 psi. The maximum design stress was equal to 980 psi and was calculated with the following input into the design equations given in Section 2.2:

WLF = 0.923 per panel $\qquad$ w_p = 53.4 in
M_{WL} = 1993 in-kips $\qquad$ t = 14.5 in
M_{LL} = 1840 in-kips $\qquad$ S_y = 1871 in^3
M_{DL} = 0 in-kips $\qquad$ f_b = 0.98 ksi = 980 psi

From the stress comparison, it can be concluded that the finite element result compares well with the design stress computed from the design manual [1].

3.2.5 Results and discussion

Figs. 3.5 to 3.10 illustrate the results of the parametric study performed on the Angelica Bridge for Load Cases 1 and 3. Upon varying the longitudinal modulus of elasticity of the panels from 1500 ksi to 1900 ksi, for Load Case 1, the maximum panel edge deflection decreased by about 15% (Fig. 3.5(a)), maximum midspan stiffener beam bending stress decreased by about 8%, and maximum panel bending stress increased by about 8% (Table

3.1). For Load Case 3, the maximum panel edge deflection decreased by about 15% (Fig. 3.5(b)), maximum midspan stiffener beam bending stress decreased by 12%, and maximum panel bending stress increased by about 6% (Table 3.1). Thus, the longitudinal modulus of elasticity of the panels has a significant effect on the deflection curve of the bridge. Deflection curves also showed a significant change near the edges of the bridge when the curb dimensions were varied. However, the effect of curbs was insignificant as we move towards the longitudinal centerline of the bridge. Upon varying the curb height from 0 to 21 inches, for Load Cases 1 and 3, the outer panel edge deflection decreased by about 80% (Fig. 3.6), but towards the center of the bridge, the difference was less than 5%. The stiffener beam bending stress towards the bridge edge increased by about 45% (Table 3.1), but the difference towards the center of the bridge was about 10%. The maximum panel bending stress decreased by about 7% with the addition of curbs (Table 3.1). Hence, we can conclude that the curbs stiffen the edges of the bridge but do not have significant effect on the deflections or stresses towards the center of the bridge.

The variation of transverse modulus of elasticity of the panels from 220 ksi to 260 ksi did not alter the deflections or stresses of the panels or stiffener beams significantly for both Load Cases (Fig. 3.7, Table 3.1). Upon varying the modulus of elasticity of the stiffener beams from 1450 ksi to 1800 ksi, the changes in the panel edge deflections were insignificant (Fig. 3.8), but the maximum stiffener beam bending stress increased by 20% for Load Case 1, and 15% for Load Case 3 (Table 3.1). The corresponding increase in stiffener beam bending stress, upon varying the stiffener beam cross-section (from 6.75in.x4.25in. to 12in.x7.5in.), is 21% for Load Case 1, and 20% for Load Case 3 (Table 3.1). When the supports of the bridge were changed from simple supports to fixed supports, the difference

between the deflection curves was significant (Fig. 3.10). In reality, the bridge may have

some rotational fixity at the abutments; hence, the supports usually act between the simply

supported and fixed end conditions. Since the experimental deflections, in general, were

lesser than the analytical results, this may be due to some effect from the rotational restraints,

if any, at the abutments or/and due to the assumed material properties in the model.

Deflection curves for all the other load cases show that the analytical results are comparable

with the experimental observations (Figs. 3.11 to 3.14).
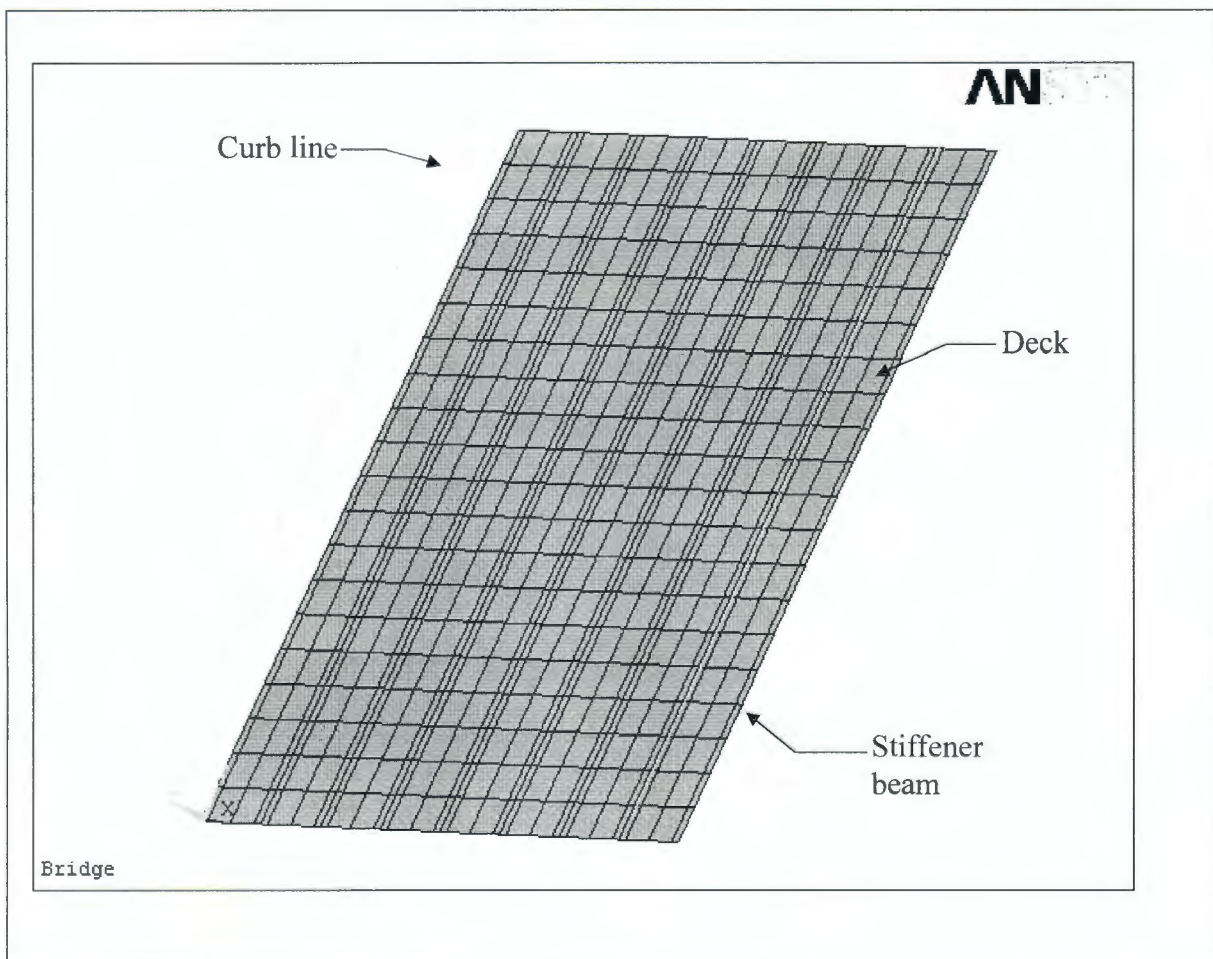
Fig. 3.4 Analytical model of the Angelica Bridge.
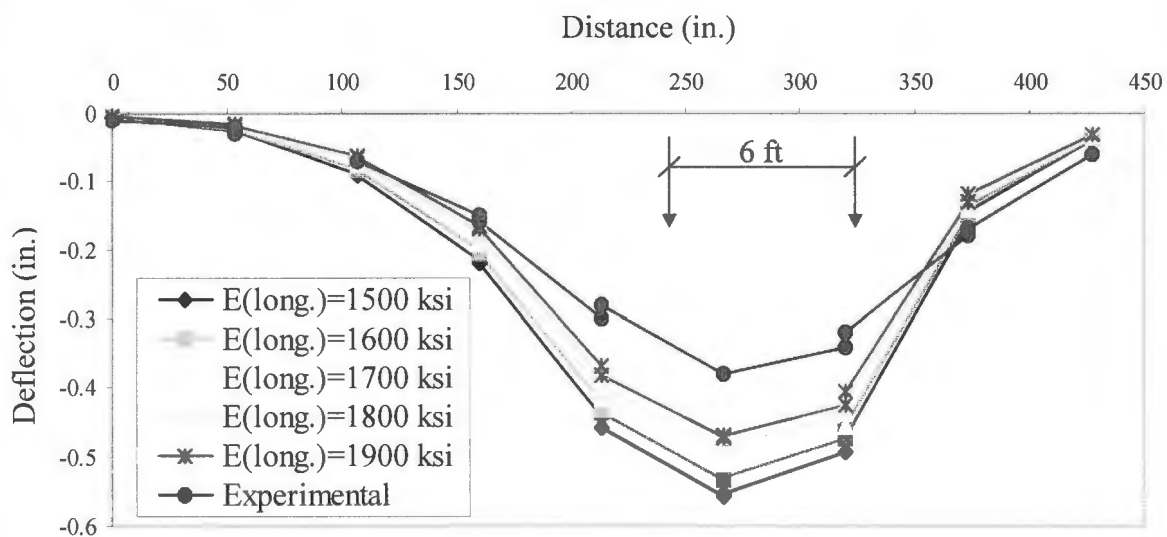
(a) Load Case 1.

(b) Load Case 3.

Fig. 3.5 Comparison of deck analytical and experimental deflection upon changing deck longitudinal modulus of elasticity for Load Cases 1 and 3.

(a) Load Case 1.
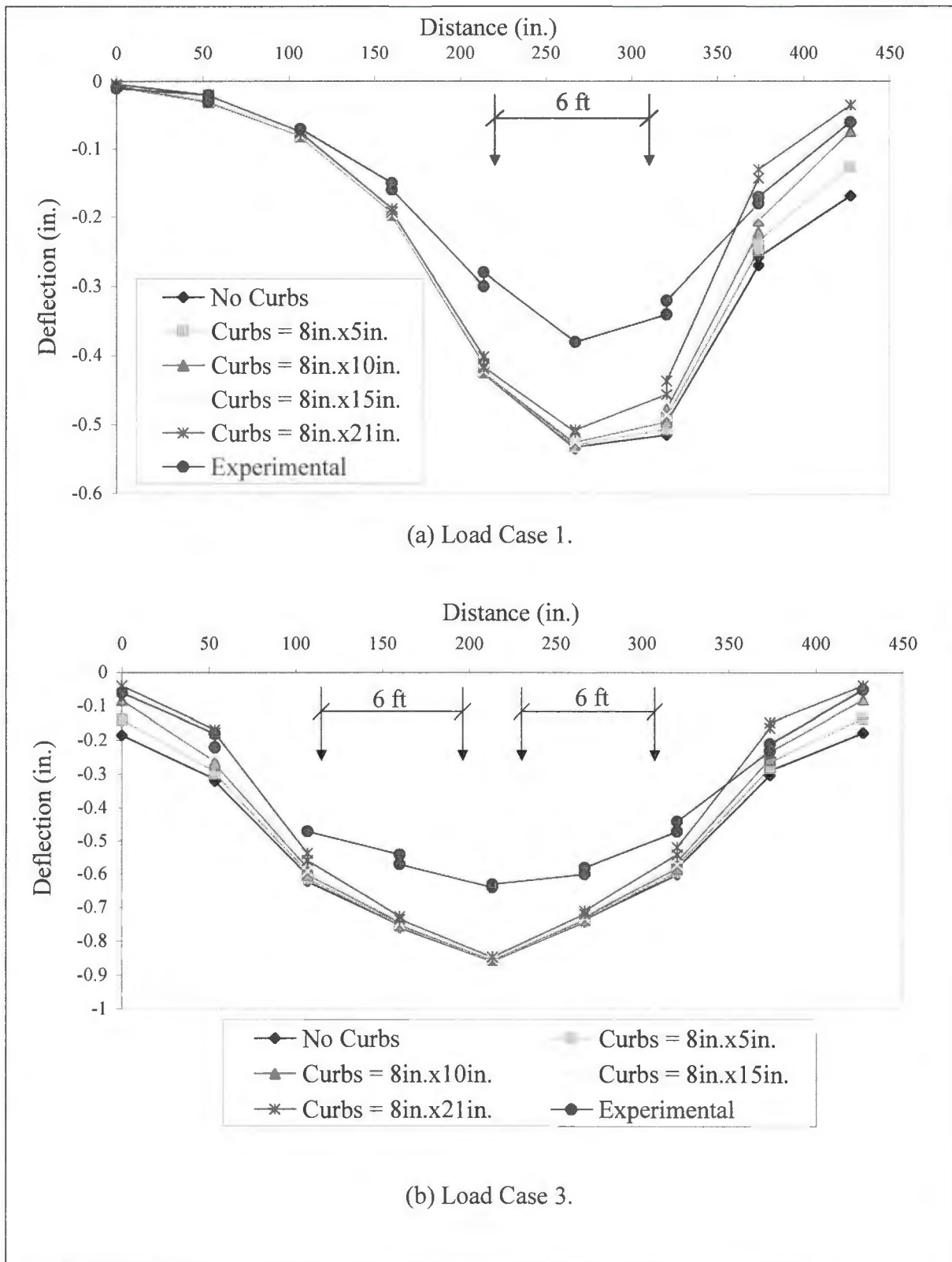
(b) Load Case 3.

Fig. 3.6 Comparison of deck experimental and analytical deflection upon varying curb height for Load Cases 1 and 3.

(a) Load Case 1.
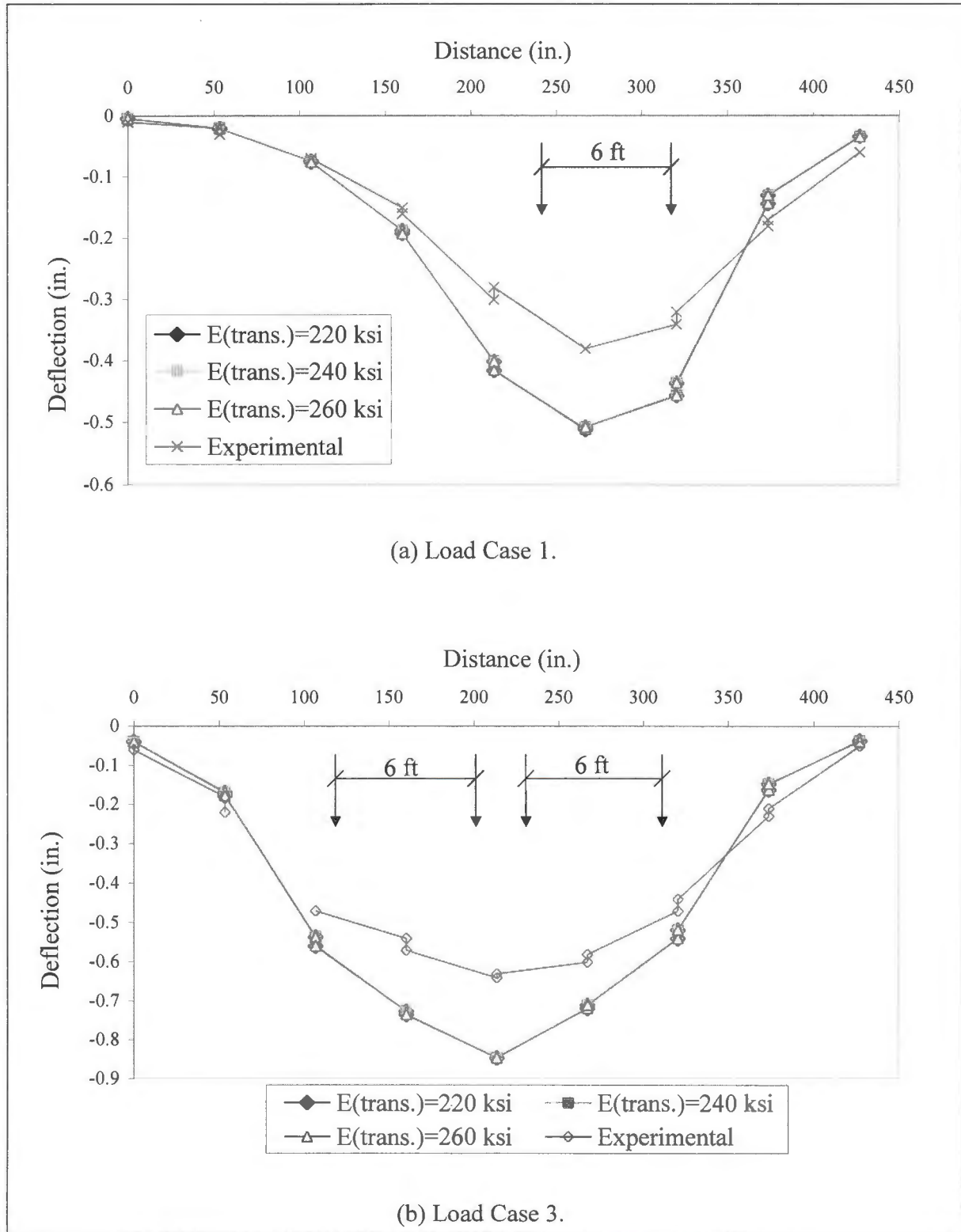
(b) Load Case 3.

Fig. 3.7 Comparison of deck experimental and analytical deflection upon changing deck transverse modulus of elasticity for Load Cases 1 and 3.
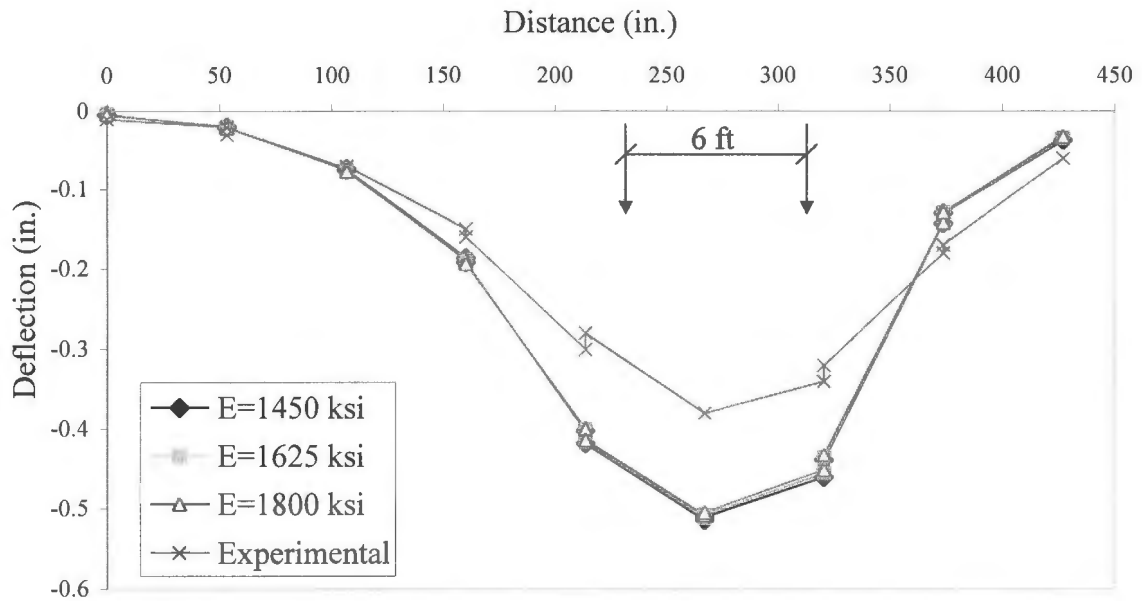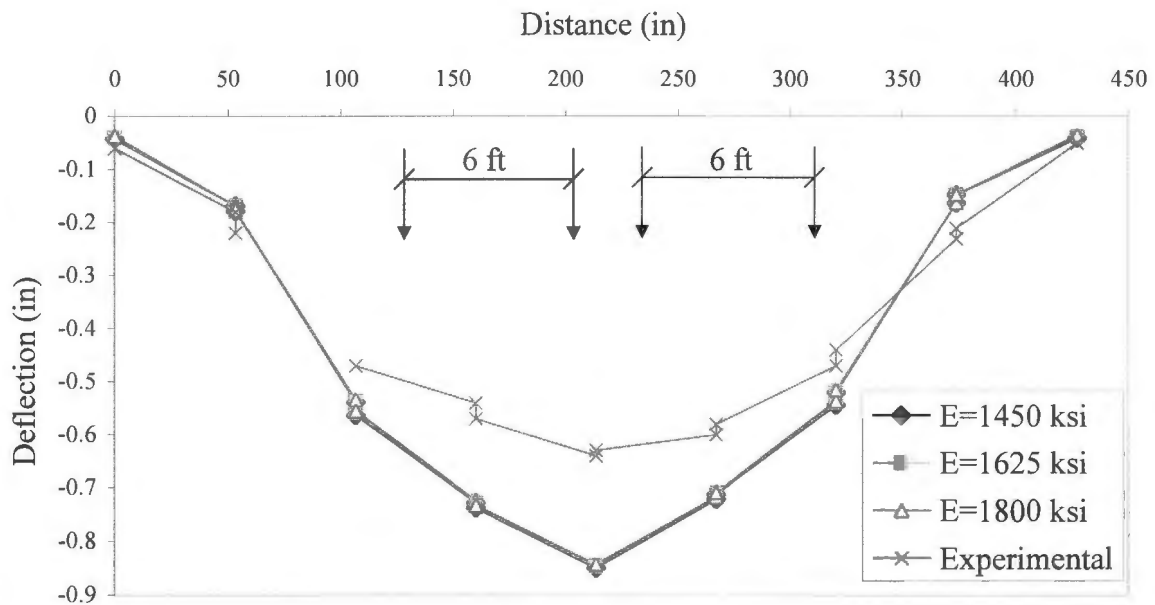
(a) Load Case 1.

(b) Load Case 3.

Fig. 3.8 Comparison of deck experimental and analytical deflection upon changing modulus of elasticity of stiffener beams for Load Cases 1 and 3.

(a) Load Case 1.
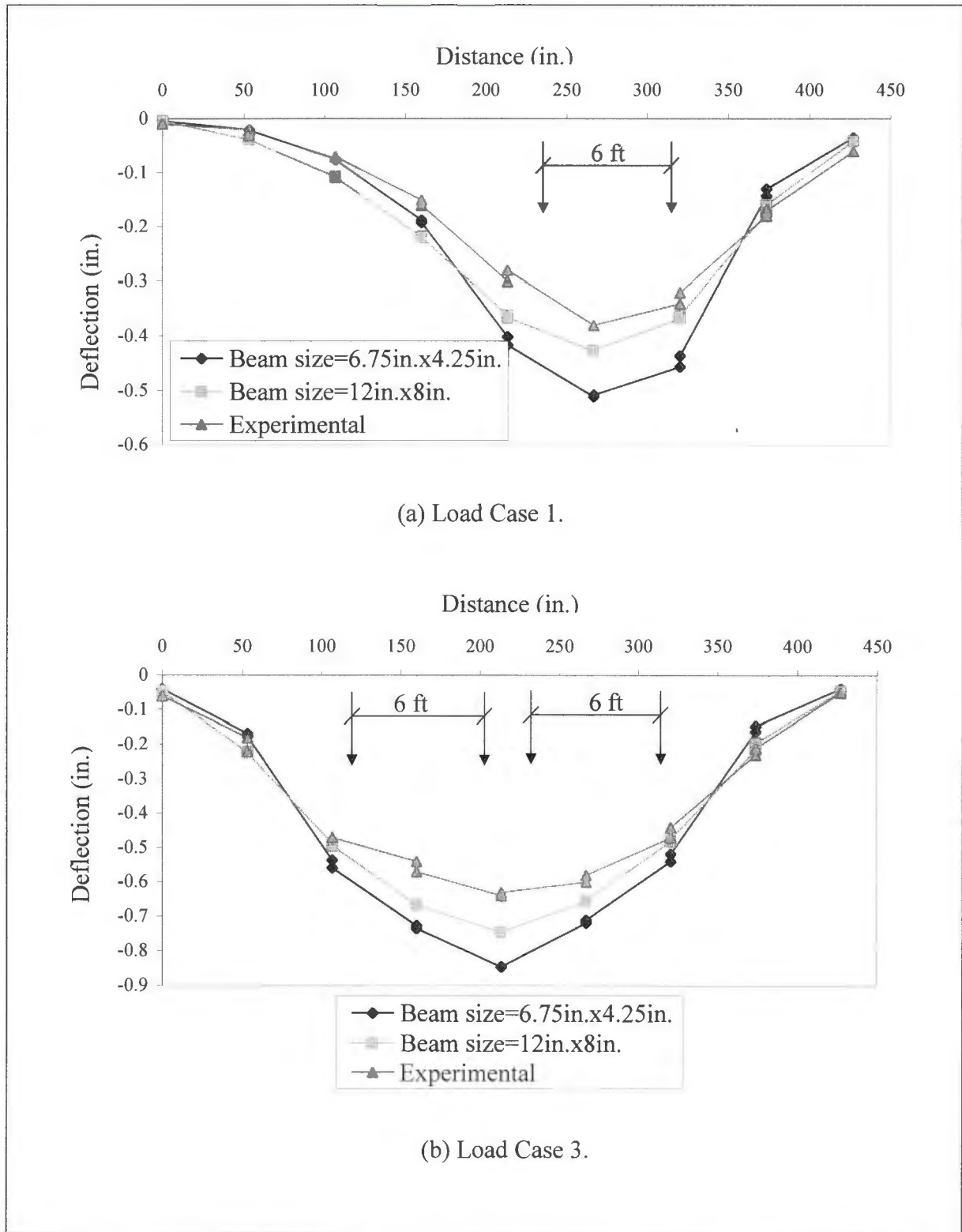
(b) Load Case 3.

Fig. 3.9 Comparison of deck experimental and analytical deflection upon changing stiffener beam cross-section dimension for Load Cases 1 and 3.

(a) Load Case 1.
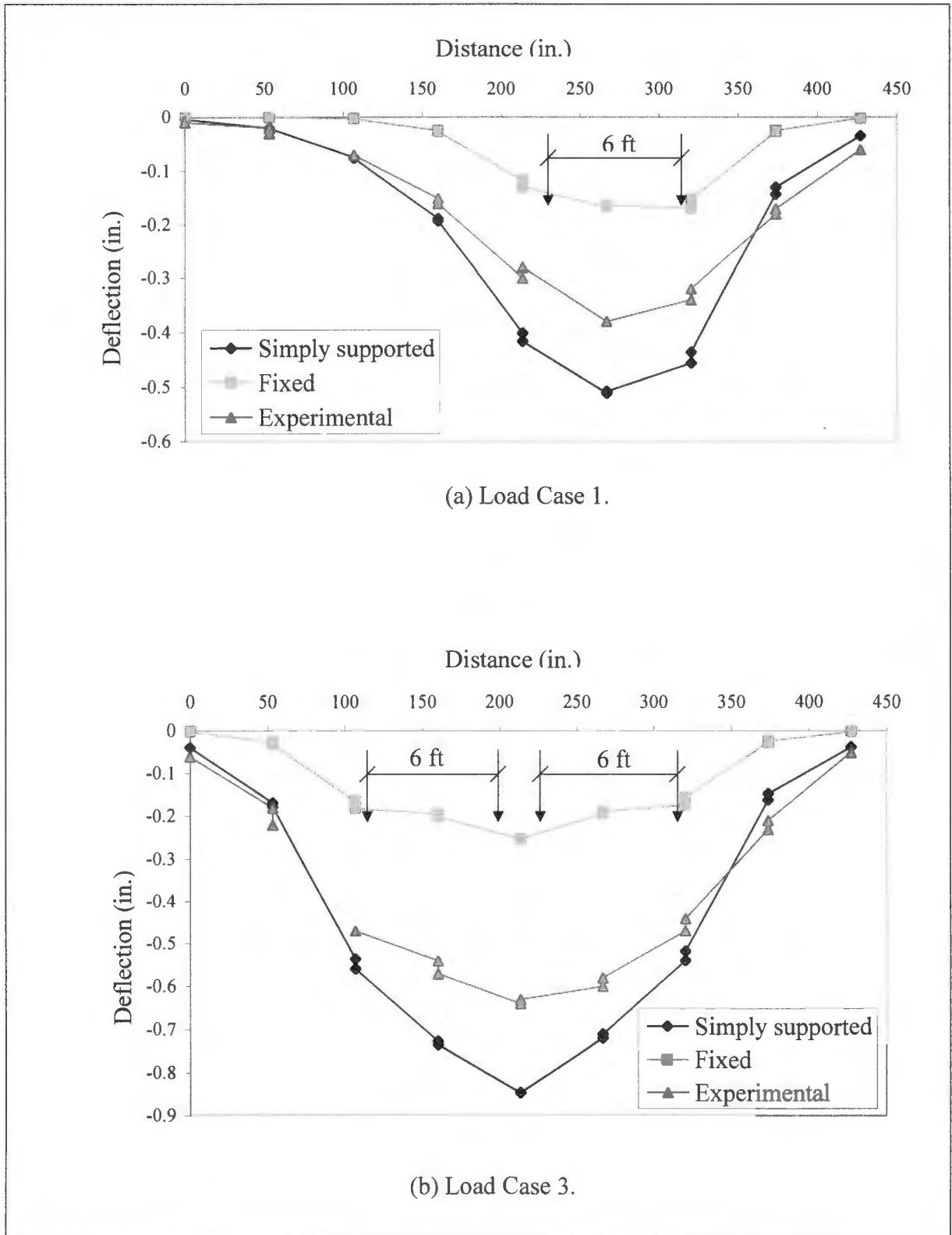
(b) Load Case 3.

Fig. 3.10 Comparison of deck experimental and analytical deflection upon changing support conditions for Load Cases 1 and 3.

Fig. 3.11 Comparison of deck experimental and analytical deflection for Load Case 2.
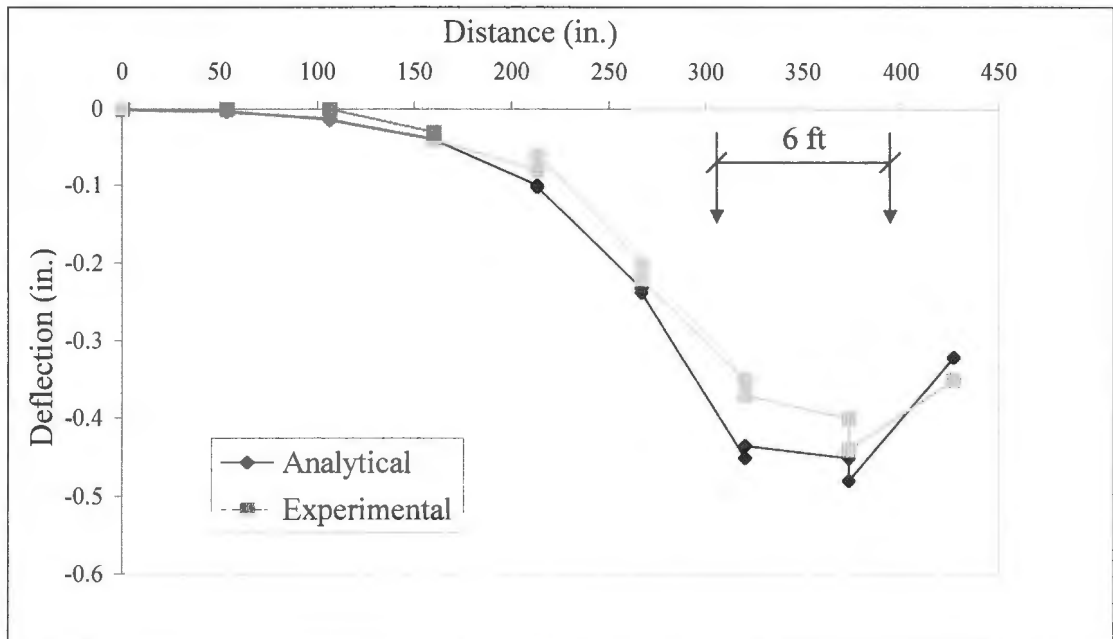
Fig. 3.12 Comparison of deck experimental and analytical deflection for Load Case 4.

Fig. 3.13 Comparison of deck experimental and analytical deflection for Load Case 5.

Fig. 3.14 Comparison of deck experimental and analytical deflection for Load Case 6.

Table 3.1 Maximum bending stresses at midspan for panel and stiffener beam (Angelica Bridge).

Criteria		Maximum stiffener beam stress (ksi)		Maximum panel stress (ksi)	
		Load Case 1	Load Case 3	Load Case 1	Load Case 3
Longitudinal elasticity of panels	E=1500 ksi	1.17	1.14	0.53	0.86
	E=1600 ksi	1.15	1.10	0.55	0.87
	E=1700 ksi	**1.12**	**1.06**	**0.56**	**0.88**
	E=1800 ksi	1.10	1.02	0.57	0.89
	E=1900 ksi	1.07	0.99	0.57	0.90
Curb height	No curbs (a)	0.62	0.76	0.60	0.89
	(b)	0.98	0.95		
	8in.x5in.(a)	0.68	0.82	0.60	0.89
	(b)	1.01	0.96		
	8in.x10in.(a)	0.73	0.87	0.59	0.89
	(b)	1.05	0.97		
	8in.x15in.(a)	0.75	0.89	0.58	0.88
	(b)	1.07	0.98		
	8in.x21in.(a)	**0.90**	**1.02**	**0.56**	**0.88**
	(b)	**1.12**	**1.06**		
Transverse elasticity of panels	E=220 ksi	1.12	1.05	0.56	0.88
	E=240 ksi	**1.12**	**1.05**	**0.56**	**0.88**
	E=260 ksi	1.12	1.07	0.56	0.88
Elasticity of stiffener beam	E=1450 ksi	1.02	0.96	0.56	0.89
	E=1625 ksi	**1.12**	**1.06**	**0.56**	**0.88**
	E=1800 ksi	1.22	1.16	0.55	0.88
Stiffener beam cross-section	**6.75in.x4.25in.**	**1.12**	**1.06**	**0.56**	**0.88**
	12in.x8in.	0.86	1.08	0.55	0.88

where (a) = region near the edge of the bridge, and
(b) = region towards midwidth of the bridge.

Note: - The **BOLD** values in the criteria column are the properties assumed for the bridge for all loadcases.

3.3 The Bolivar Bridge, NY

3.3.1 Description

The Bolivar Bridge (Fig. 3.15) is a two-lane, glulam deck bridge with six panels and three stiffener beams running transverse to the longitudinally placed deck panels. The stiffener beams are connected to the deck panels by through-bolt connections. The bridge deck is made of Douglas Fir timber. The deck panels are about 4.3 feet wide, 28 feet long, and 14.56 inches thick and are placed longitudinally between supports. These deck panels are not interconnected. The stiffener beams are connected to the deck panels by through-bolts at 6 inches in from each edge of the panel. The bridge measured about 28 feet in span, measured center-to-center of bearings. The stiffener beams have a nominal width of 6.75 inches, and a nominal thickness of 4.625 inches. The bridge has deep curbs on its edges with a cross-section dimension of 8 inches by 21.5 inches. The material properties for the analytical model were obtained from the Douglas Fir Handbook [9] for the particular grade of lamination used on the bridge. The deck panels have a longitudinal and transverse modulus of elasticity of 1416 kips-per-square inch (ksi) and 124 ksi respectively. The shear modulus of the deck panels is about 100 ksi. The stiffener beams have a longitudinal modulus of elasticity of 1625 ksi. The shear modulus of the stiffener beams is about 80 ksi. Refer to Fig. 3.16 for the design configuration of the bridge.

3.3.2 Loading

Six load tests were done on the bridge. The load test vehicles consisted of two fully loaded three-axle dump trucks with gross vehicle weights of 71,980 pounds (Truck 12) and 68,840 pounds (Truck 18). The rear axles on Truck 12 weighed about 53,980 pounds while

(a) Elevation view of the Bolivar Bridge.

(b) End view of the Bolivar Bridge.

Fig. 3.15 The Bolivar Bridge in NY.

the rear axles on Truck 18 weighed about 51,630 pounds. The rear axles weighed about 75% of the respective gross vehicle weights. The vehicles were positioned longitudinally on the bridge so that the two rear axles were centered on the midspan of the bridge. Since this is a short-span bridge, the front axle of the trucks was outside the bridge for all load cases. The transverse vehicle track width, measured center-to-center of the rear tires, was 6 feet. For Load Cases 1 and 4, Truck 12 was placed on the east lane of the bridge, 24 inches and 53 inches away from the longitudinal centerline of the bridge, respectively. For Load Cases 2 and 5, Truck 18 was placed on the west lane of the bridge, 24 inches and 53 inches away from the longitudinal centerline of the bridge, respectively. Load Case 3 was a combination of Load Cases 1 and 2, while Load Case 6 was a combination of Load Cases 4 and 5. Fig. 3.17 shows the configuration of the trucks and the load positions on the bridge.

3.3.3 Parametric study

A parametric study on the Bolivar Bridge was done for Load Cases 1 and 3. The input data described in Section 3.2.1 was used for the initial run on the program. The material properties of the deck were then varied and a comparative plot for panel edge deflections at midspan of bridge was made. Comparison was also done with respect to the experimental data from field tests. Similar deflection comparisons were made by individually changing the curb height and end restraint. The analytical model of the bridge is shown in Fig. 3.18. Figs. 3.19 to 3.21 show the comparative plots for the full parametric study.

A comparative study of experimental values and analytical results for midspan panel edge deflections was also done for all the other load cases. The deflection plots for this comparison are shown in Figs. 3.22 to 3.25. A mesh sensitivity study was also done on the
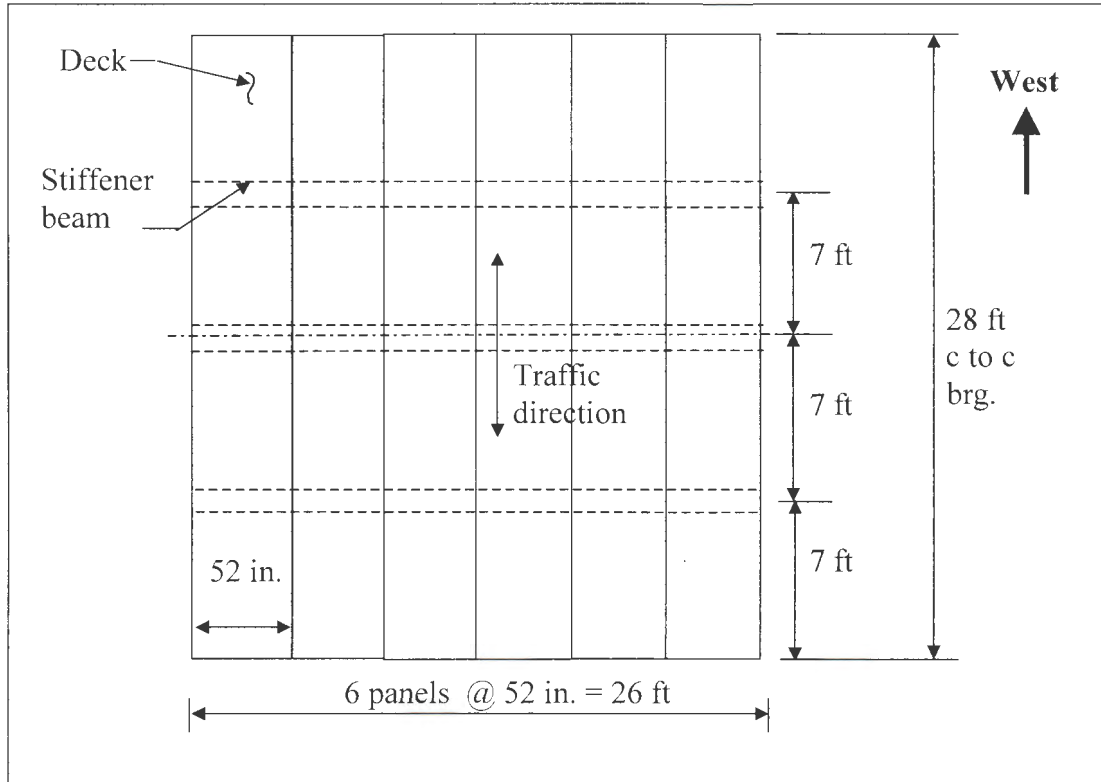
Fig. 3.16 Plan layout of the Bolivar Bridge.

analytical model of the bridge to determine the adequate size of the element to be used.

3.3.4 Comparison of maximum design stress and maximum stress from analysis

The maximum design stress for the panels on the Bolivar Bridge was computed using the procedure described in Section 2.2. The maximum longitudinal panel stress at midspan obtained using the program was 706 psi. The maximum design stress was equal to 850 psi and was calculated with the following input into the design equations given in Section 2.2:

$WLF = 0.912$ per panel $w_p = 52$ in
$M_{WL} = 1698$ in-kips $t = 14.5$ in
$M_{LL} = 1548$ in-kips $S_y = 1822$ in^3
$M_{DL} = 0$ in-kips $f_b = 0.85$ ksi $= 850$ psi

From the stress comparison, it can be concluded that the finite element result compares well with the design stress computed from the design manual [1].
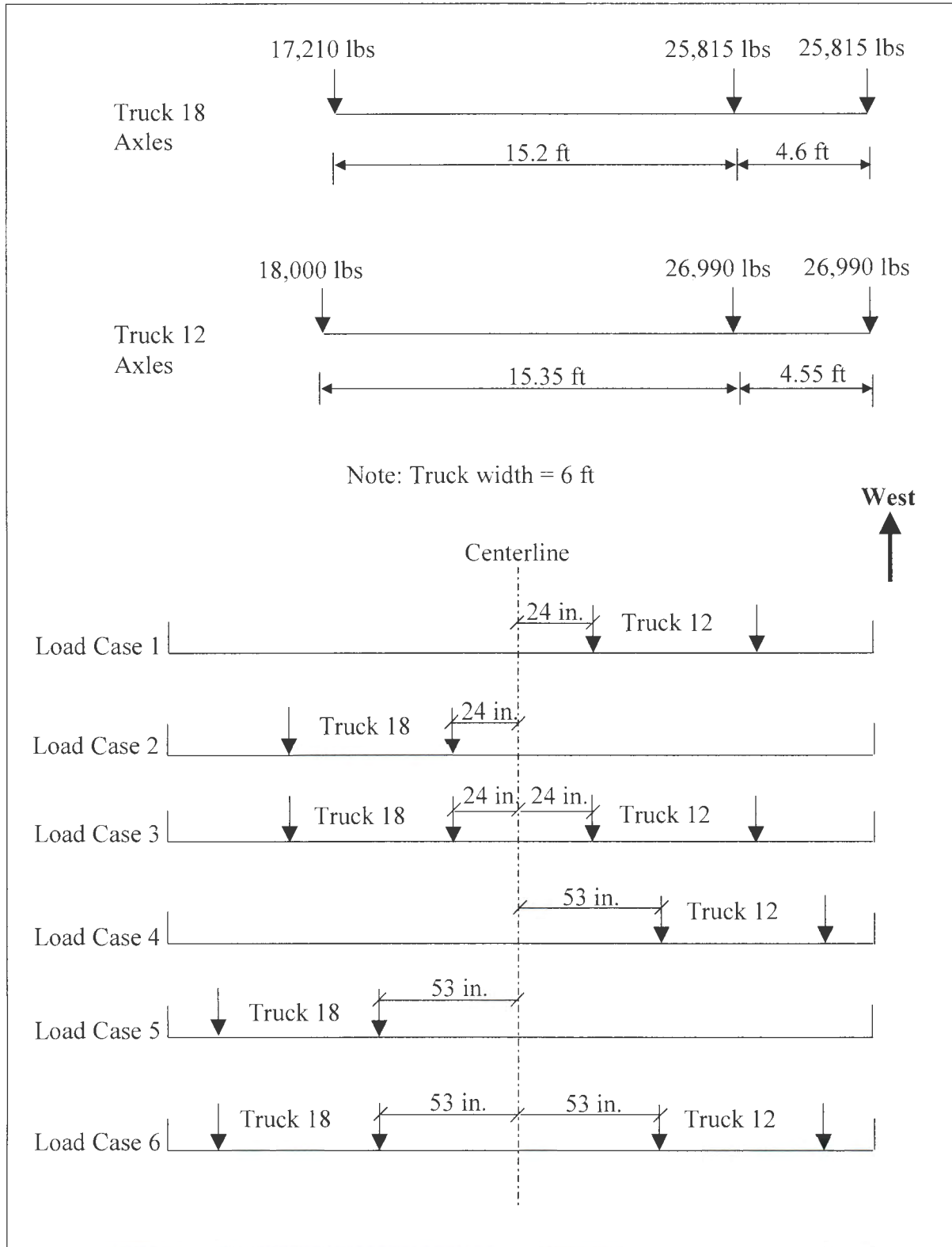
Fig. 3.17 Configuration of trucks and load positions for various load cases.

3.3.5 Results and discussion

Figs. 3.19 to 3.21 illustrate the results of the parametric study performed on the Bolivar Bridge for Load Cases 1 and 3. Upon varying the longitudinal modulus of elasticity of the panels from 1300 ksi to 1700 ksi for Load Case 1, the maximum panel edge deflection decreased by about 15% (Fig. 3.19(a)), maximum midspan stiffener beam bending stress decreased by about 14%, and maximum panel bending stress increased by about 9% (Table 3.2). For Load Case 3, the maximum panel edge deflection decreased by 17% (Fig. 3.19(b)), maximum midspan stiffener beam bending stress decreased by 16%, and maximum panel bending stress increased by about 9% (Table 3.2). The longitudinal modulus of elasticity of the panels is a significant parameter that influences the analytical behavior of the bridge.

Upon varying the curb height from 0 to 21 inches for Load Cases 1 and 3, there was a significant decrease in the outer panel edge deflection (Fig. 3.19(a)). The stiffener beam bending stress towards the bridge edge increased by about 47% for both load cases (Table 3.2), but the difference towards the center of the bridge was about 15%. The maximum panel bending stress decreased by about 7% for Load Case 1, and about 15% for Load Case 3 (Table 3.2), with the addition of curbs. Hence, we can conclude that the curbs play an important role in stiffening the edge of the bridge but they have an insignificant effect on the deflections or stresses towards the longitudinal centerline of the bridge.

When the supports of the bridge were changed from simple supports to fixed supports, the difference between the deflection curves was significant (Fig. 3.21). However, since the experimental deflections were, in general, higher than the analytical results, the effect of rotational fixity at the abutments can be assumed to be negligible. The differences between the analytical results and the experimental values may be because of assumed

material properties in the model or because of the way the connections between the deck and stiffener beams were modeled, or a combination of both factors. Deflection curves for all the other load cases show that the analytical results are comparable with the experimental observations (Figs. 3.22 to 3.25).
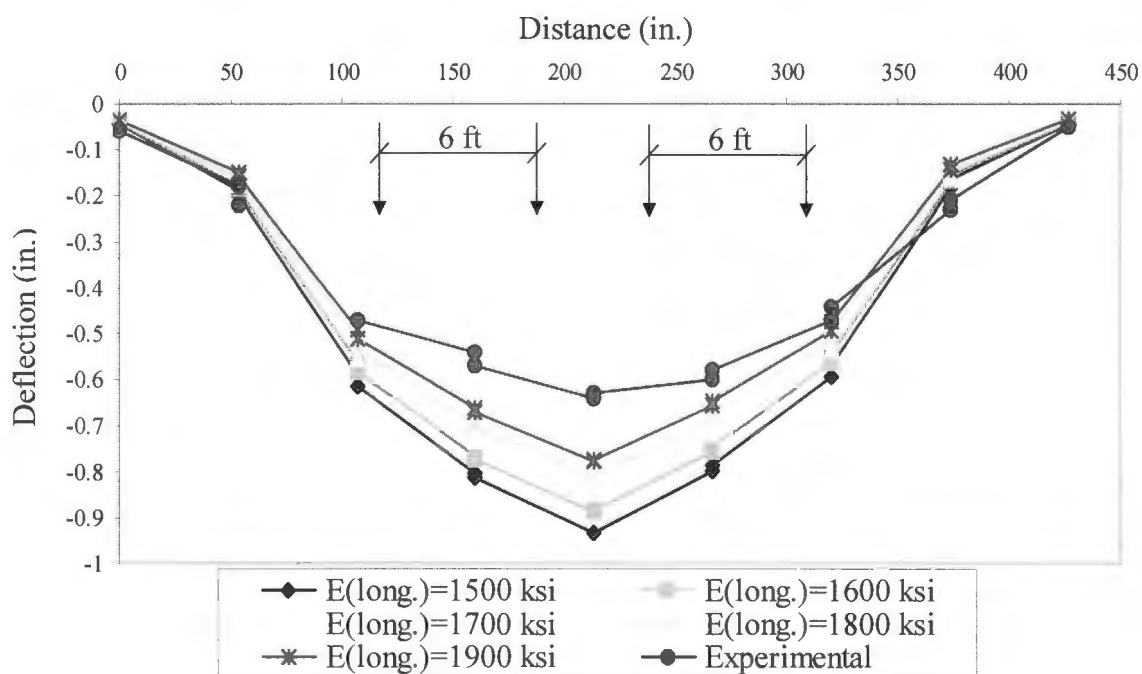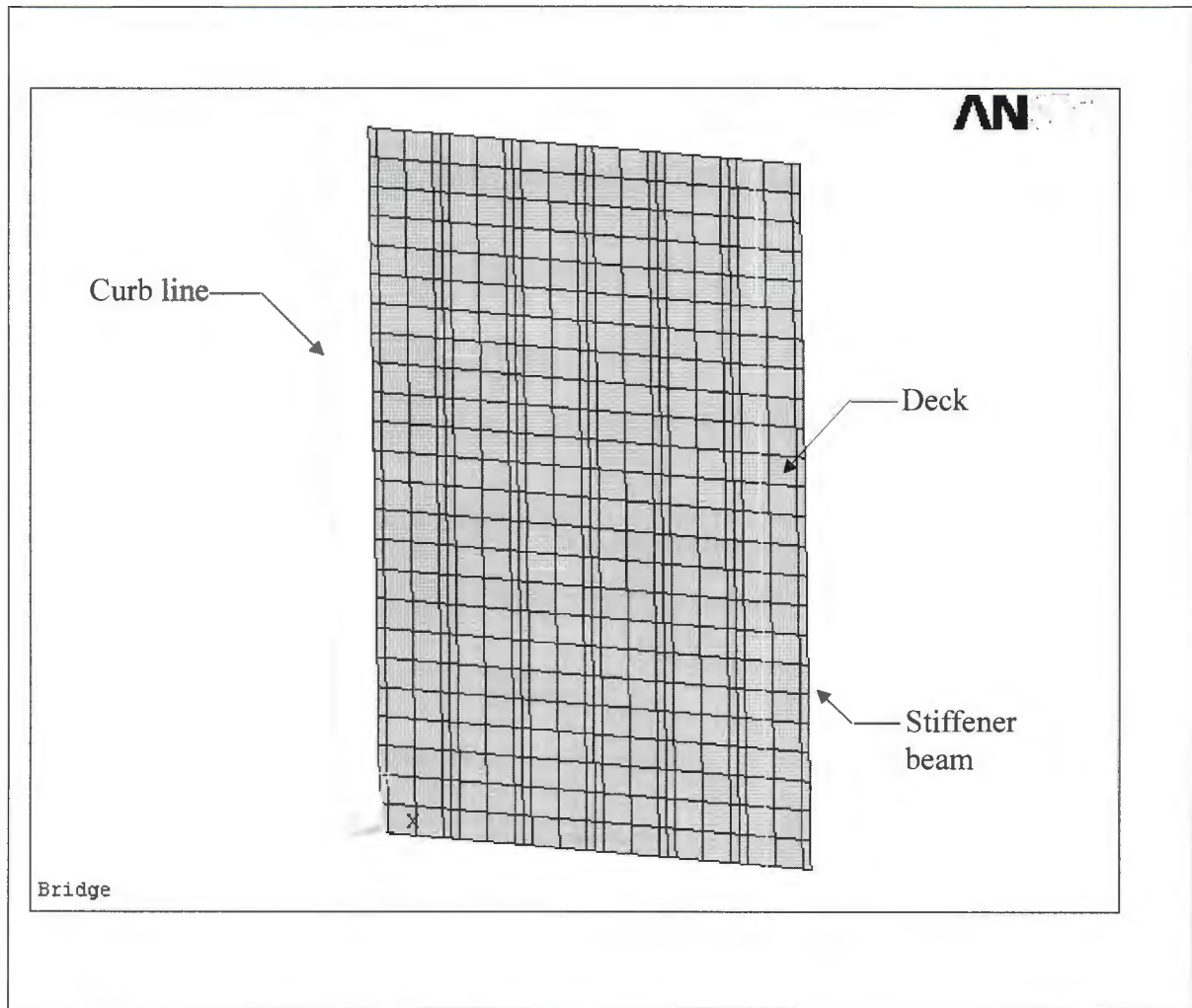

Fig. 3.18 Analytical model of the Bolivar Bridge.

(a) Load Case 1.
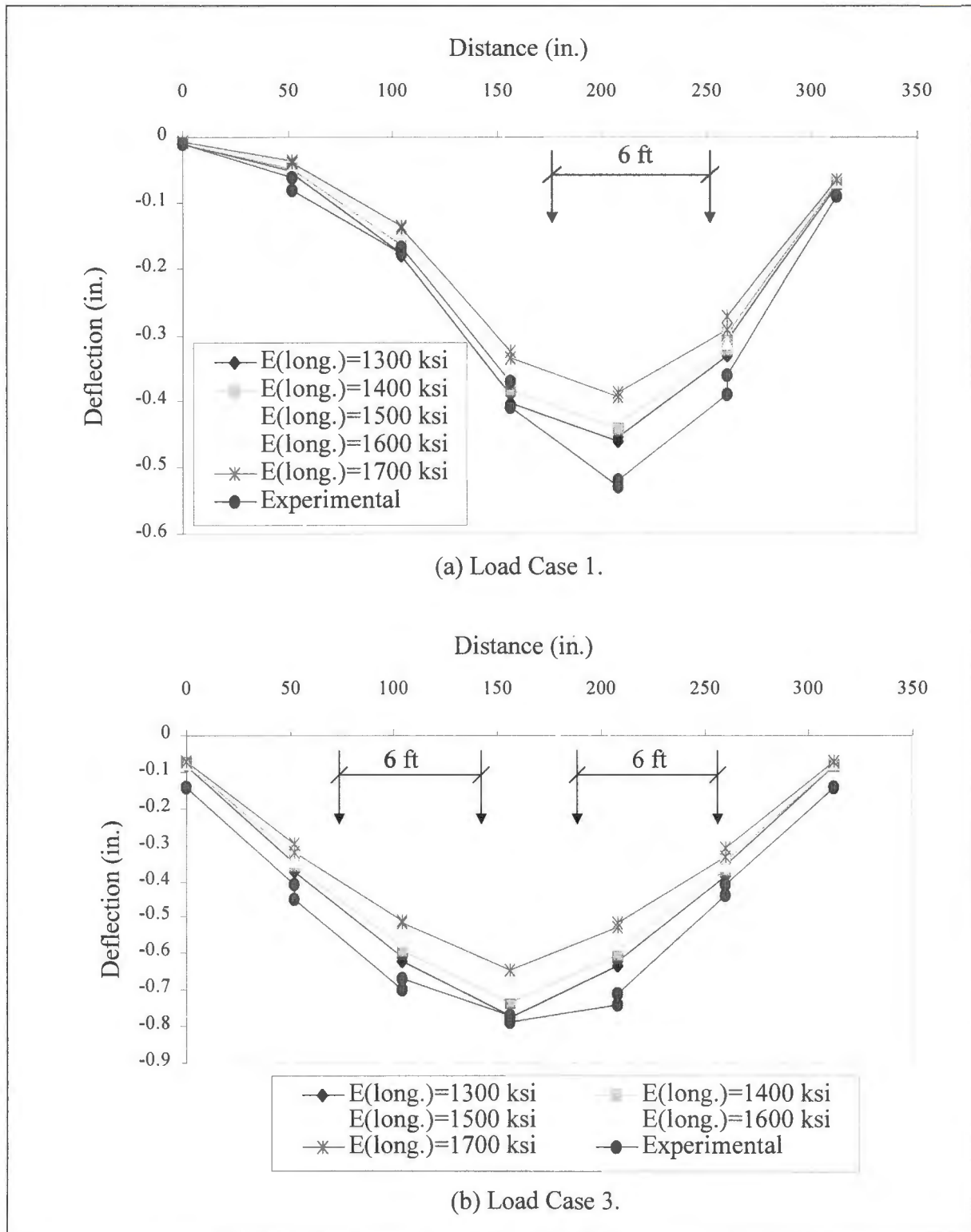
(b) Load Case 3.

Fig. 3.19 Comparison of deck analytical and experimental deflection upon changing deck longitudinal modulus of elasticity for Load Cases 1 and 3.

(a) Load Case 1.
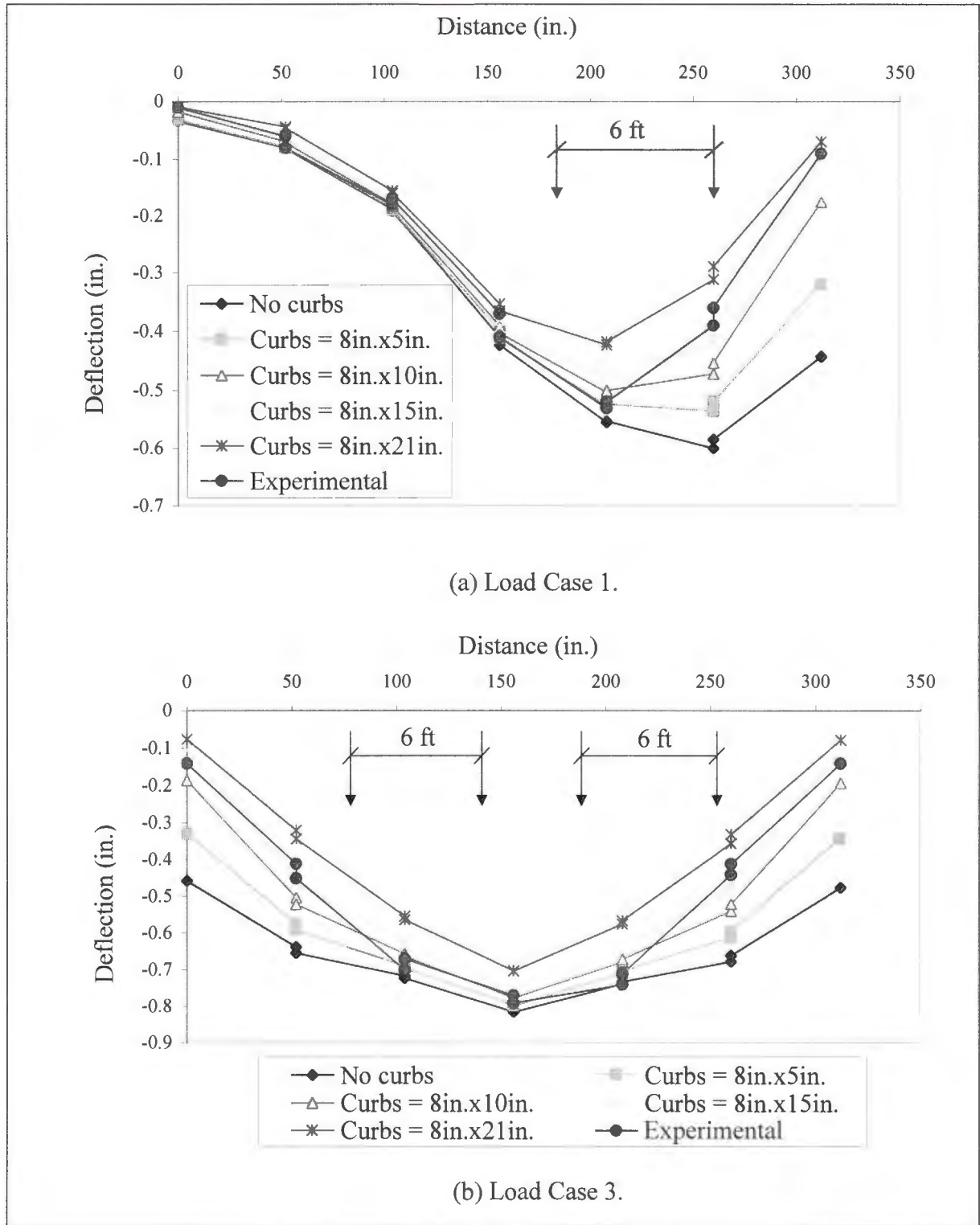
(b) Load Case 3.

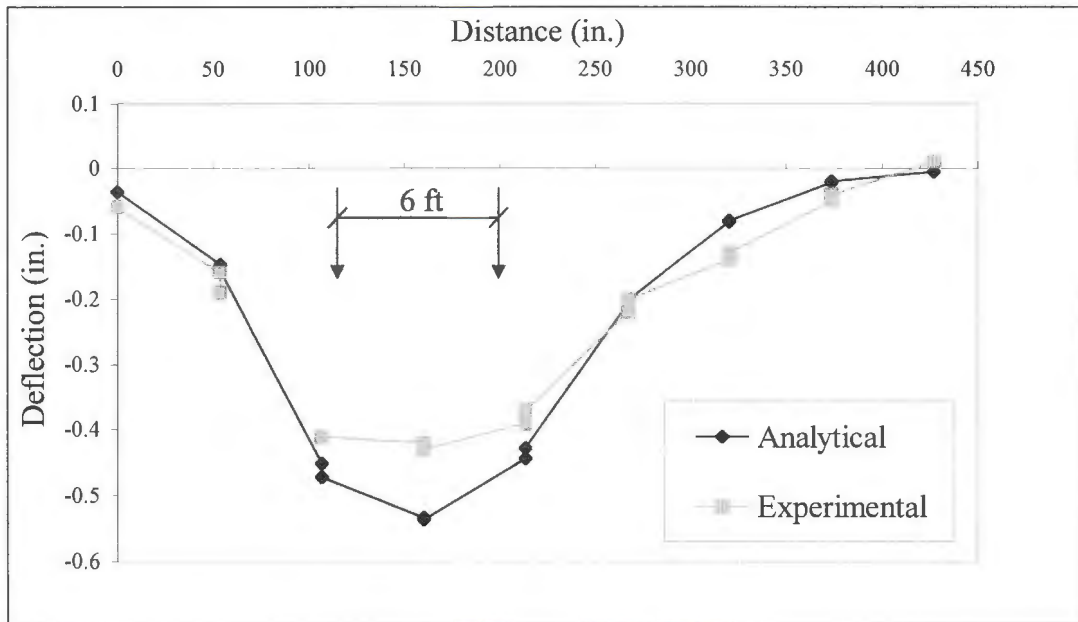**Fig. 3.20 Comparison of deck experimental and analytical deflection upon varying curb height for Load Cases 1 and 3.**
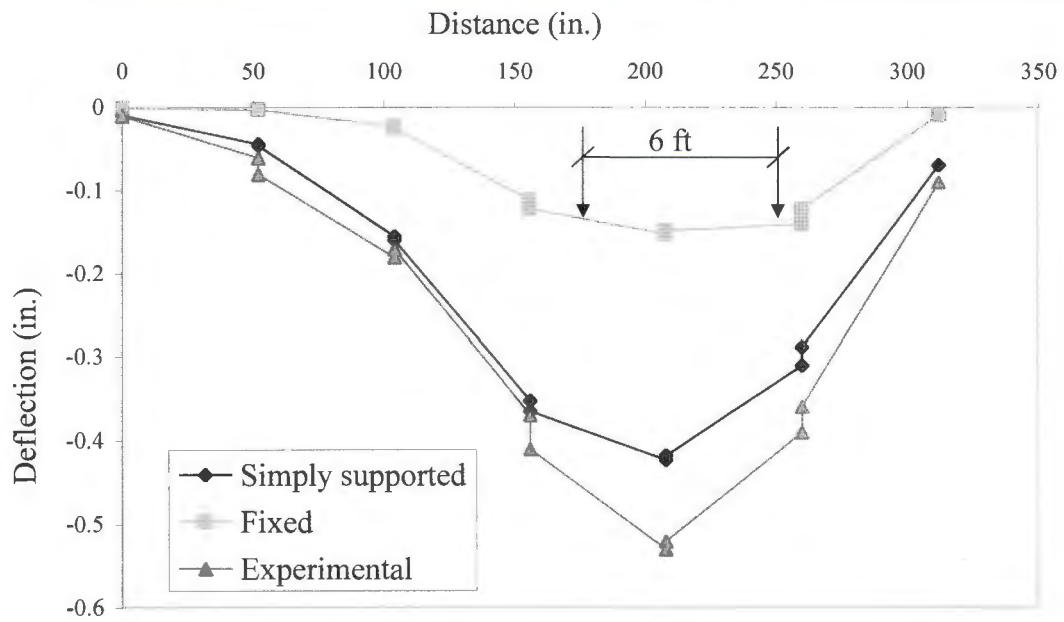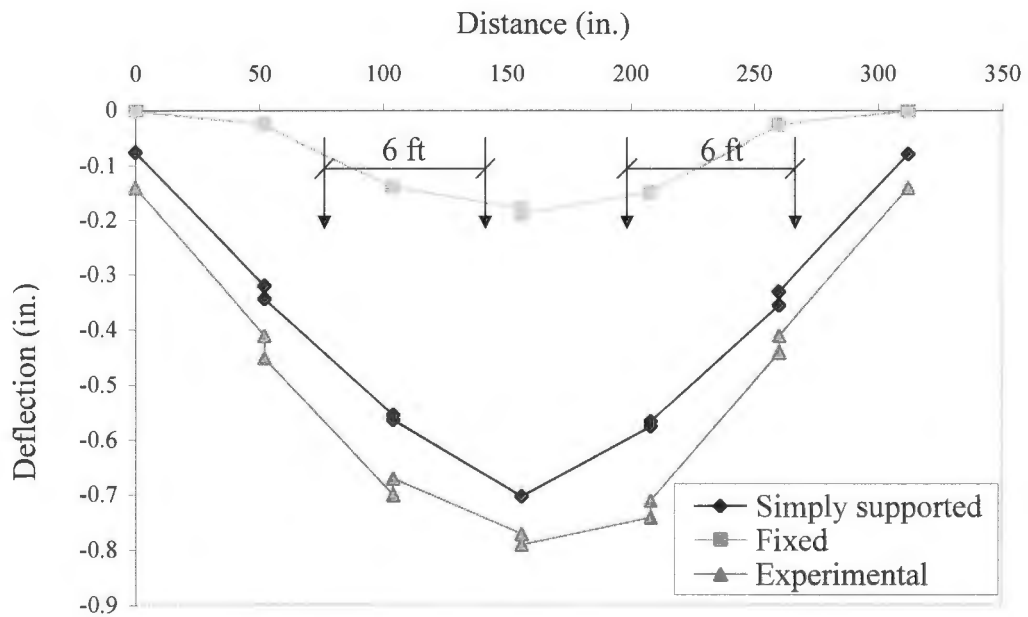
(a) Load Case 1.

(b) Load Case 3.

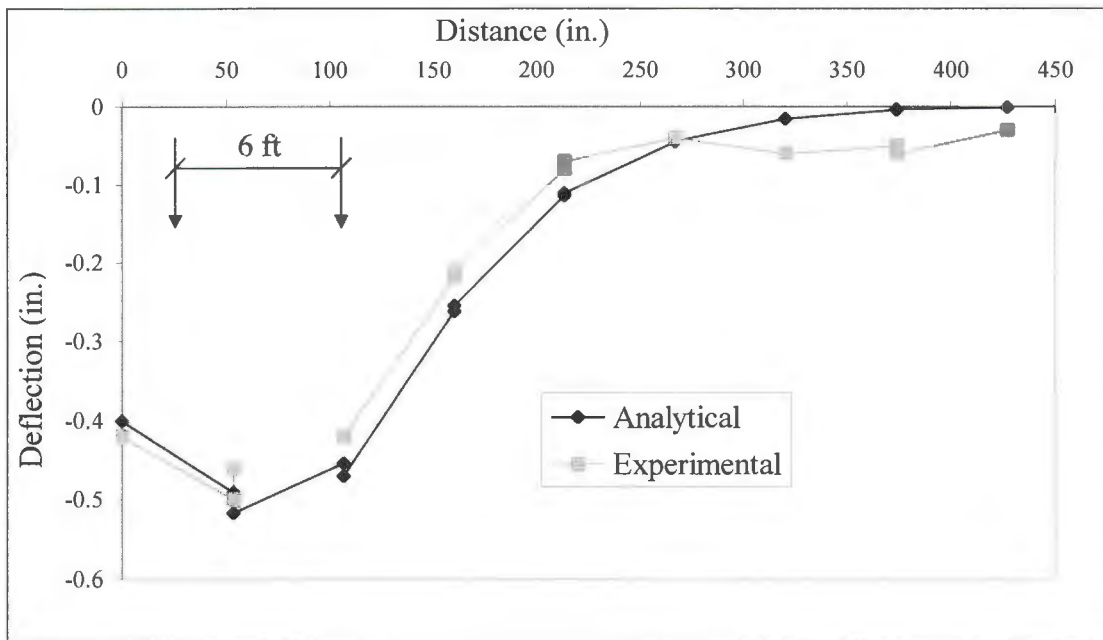**Fig. 3.21 Comparison of deck experimental and analytical deflection upon changing support conditions for Load Cases 1 and 3.**
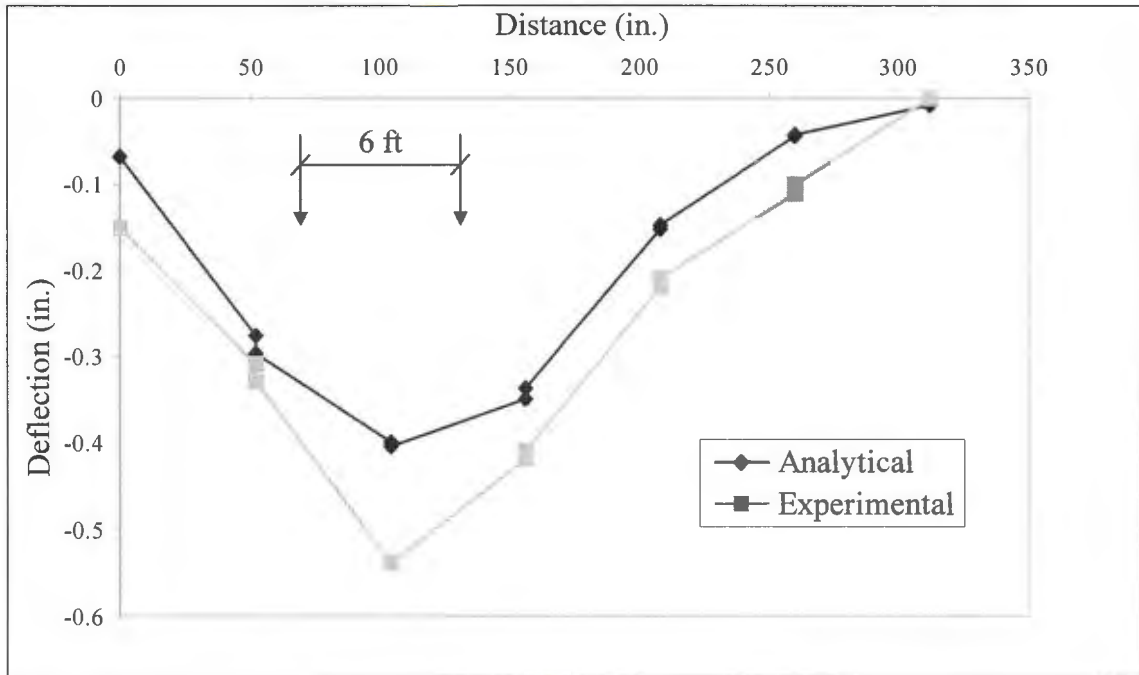
Fig. 3.22 Comparison of deck experimental and analytical deflection for Load Case 2.

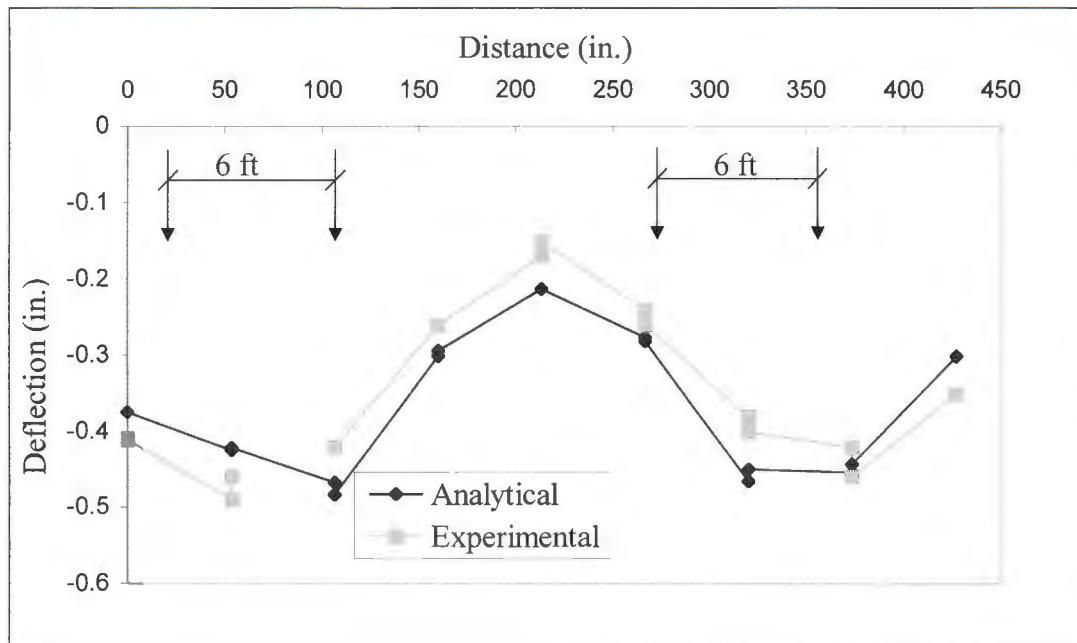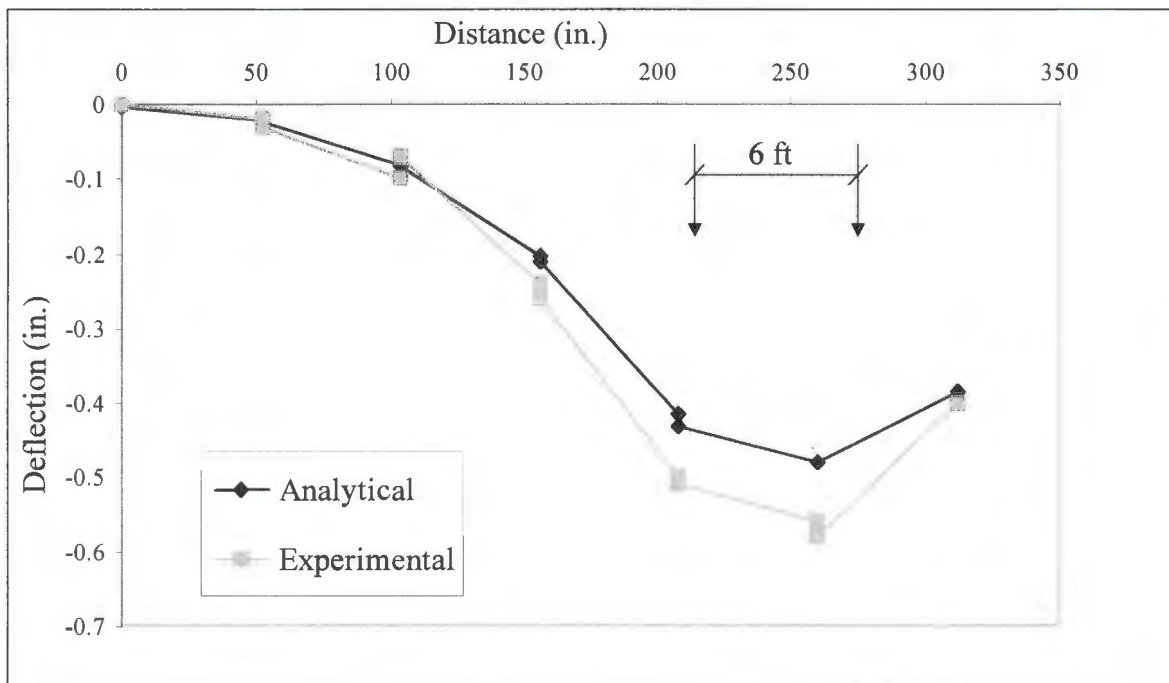**Fig. 3.23 Comparison of deck experimental and analytical deflection for Load Case 4.**
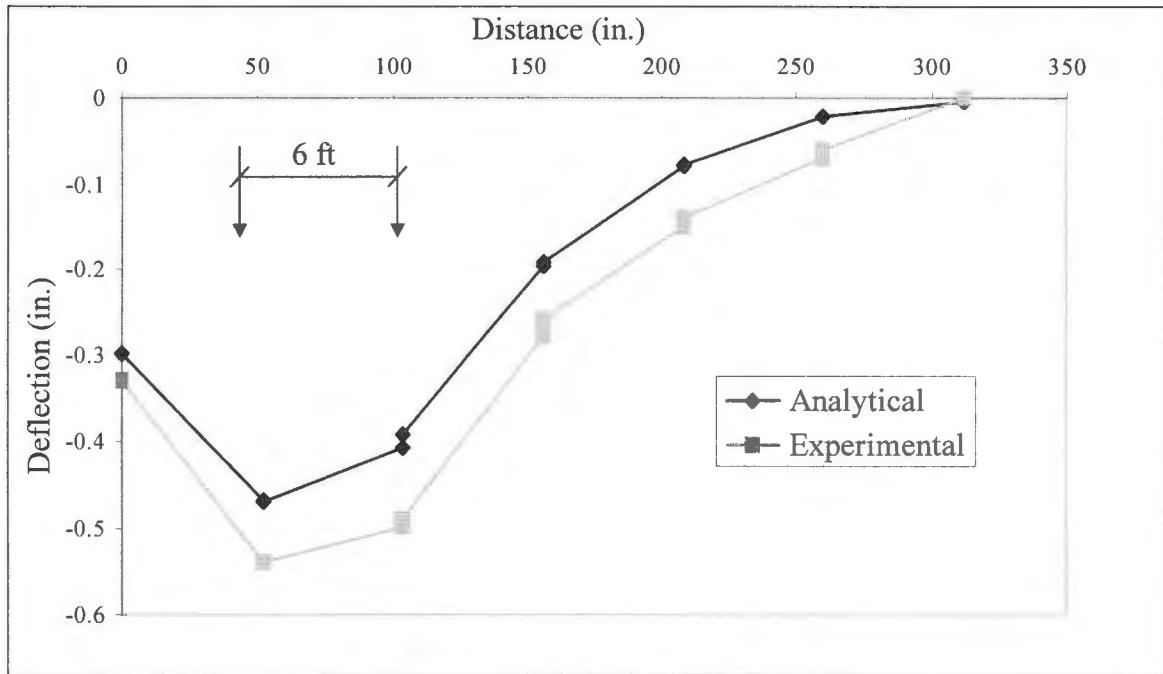
Fig. 3.24 Comparison of deck experimental and analytical deflection for Load Case 5.
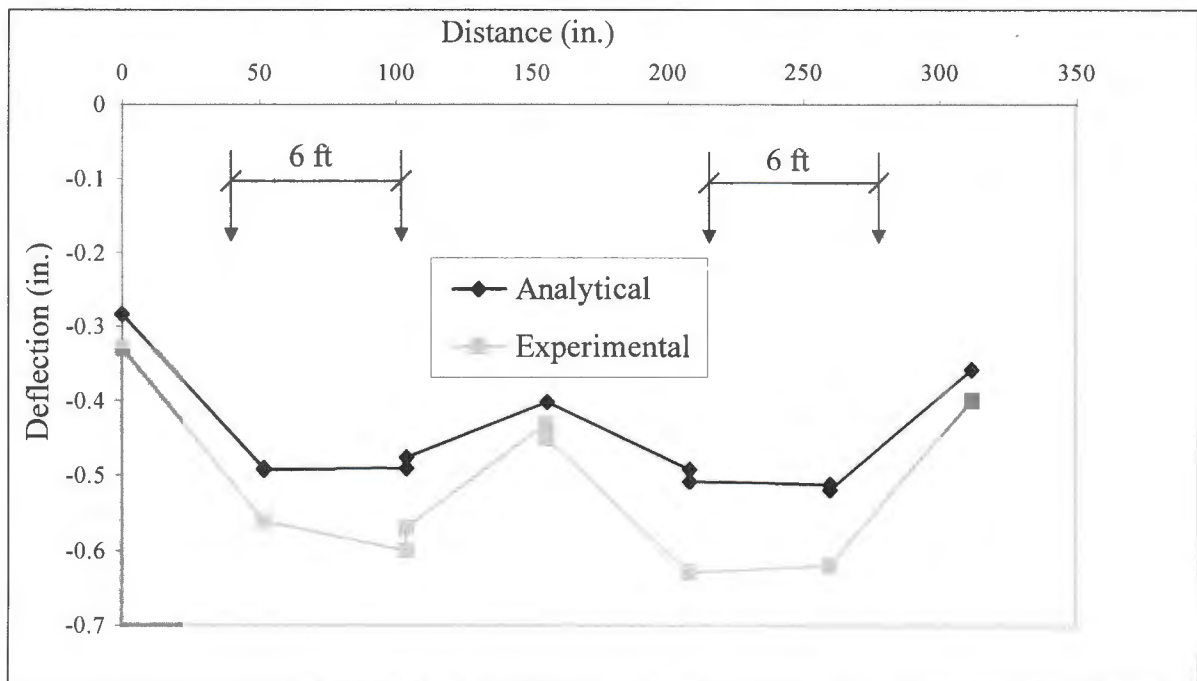
Fig. 3.25 Comparison of deck experimental and analytical deflection for Load Case 6.

Table 3.2 Maximum bending stresses at midspan for panel and stiffener beam (Bolivar Bridge).

Criteria		Maximum stiffener beam stress (ksi)		Maximum panel stress (ksi)	
		Load Case 1	Load Case 3	Load Case 1	Load Case 3
Longitudinal elasticity of panels	E=1300 ksi	0.77	1.31	0.45	0.67
	E=1400 ksi	0.75	1.25	0.46	0.69
	E=1500 ksi	**0.73**	**1.20**	**0.47**	**0.71**
	E=1600 ksi	0.69	1.15	0.48	0.72
	E=1700 ksi	0.66	1.10	0.49	0.74
Curb height	No Curbs (a)	0.31	0.70	0.50	0.83
	(b)	0.64	0.84		
	8in.x5in. (a)	0.36	0.74	0.49	0.81
	(b)	0.66	0.88		
	8in.x10in. (a)	0.42	0.84	0.49	0.79
	(b)	0.68	0.91		
	8in.x15in. (a)	0.44	0.98	0.48	0.74
	(b)	0.70	0.99		
	8in.x21in. (a)	**0.46**	**0.99**	**0.47**	**0.71**
	(b)	**0.73**	**1.20**		

where   (a) = region near the edge of the bridge, and
           (b) = region towards the midwidth of the bridge.

<u>Note:</u> - The **BOLD** values in the criteria column are the properties assumed for the bridge for all loadcases.

4. GLUED LAMINATED GIRDER BRIDGES

4.1 Description of girder bridges

Glued laminated (glulam) bridges are the most common type of timber bridges. The spans of these bridges range from 20 to 100 feet. In this type of bridge, the deck panels are laid transverse to the girders that run between supports. The deck panels consist of a series of laminated lumbers that are placed on edge and glued together on their wide faces. The panels are normally about 4 feet in width and 5 to 7 inches in thickness. The girders are also glued laminated are usually 5 to 12 inches in width with depth to width ratios of 2 to 1 or greater. Lag bolts are used to connect the girders to the deck panels and this is responsible for the composite action between the deck and the girder. The bridge railing system consists of treated timber posts and a glued laminated rail, faced with a galvanized steel w-beam. The approach guardrail system us usually treated timber posts with galvanizes steel w-beam. A generic photo of a glued laminated girder bridge is shown in Fig. 4.1. Fig. 4.2 illustrates the configuration of a typical glulam girder bridge.

The deck panels are manufactured of vertically laminated lumber and the loads act parallel to the wide face of the laminations. The two basic types of glulam decks are the non-interconnected deck and the doweled deck. Non-interconnected decks have no mechanical connection between adjacent panels. Doweled decks are interconnected with steel dowels to distribute the loads between adjacent panels. Both deck types are stronger and stiffer than conventional nail-laminated lumber or plank decks, resulting in longer deck spans, increased spacing of supporting girders, and reduced live load deflection. The glulam panels can also

(a) Single span glulam girder bridge.

(b) Multi-span glulam girder bridge.

Fig. 4.1 Generic photo of a glulam girder bridge.

(a) Side view.

(b) Cross-section view.

(c) Cutaway plan view.
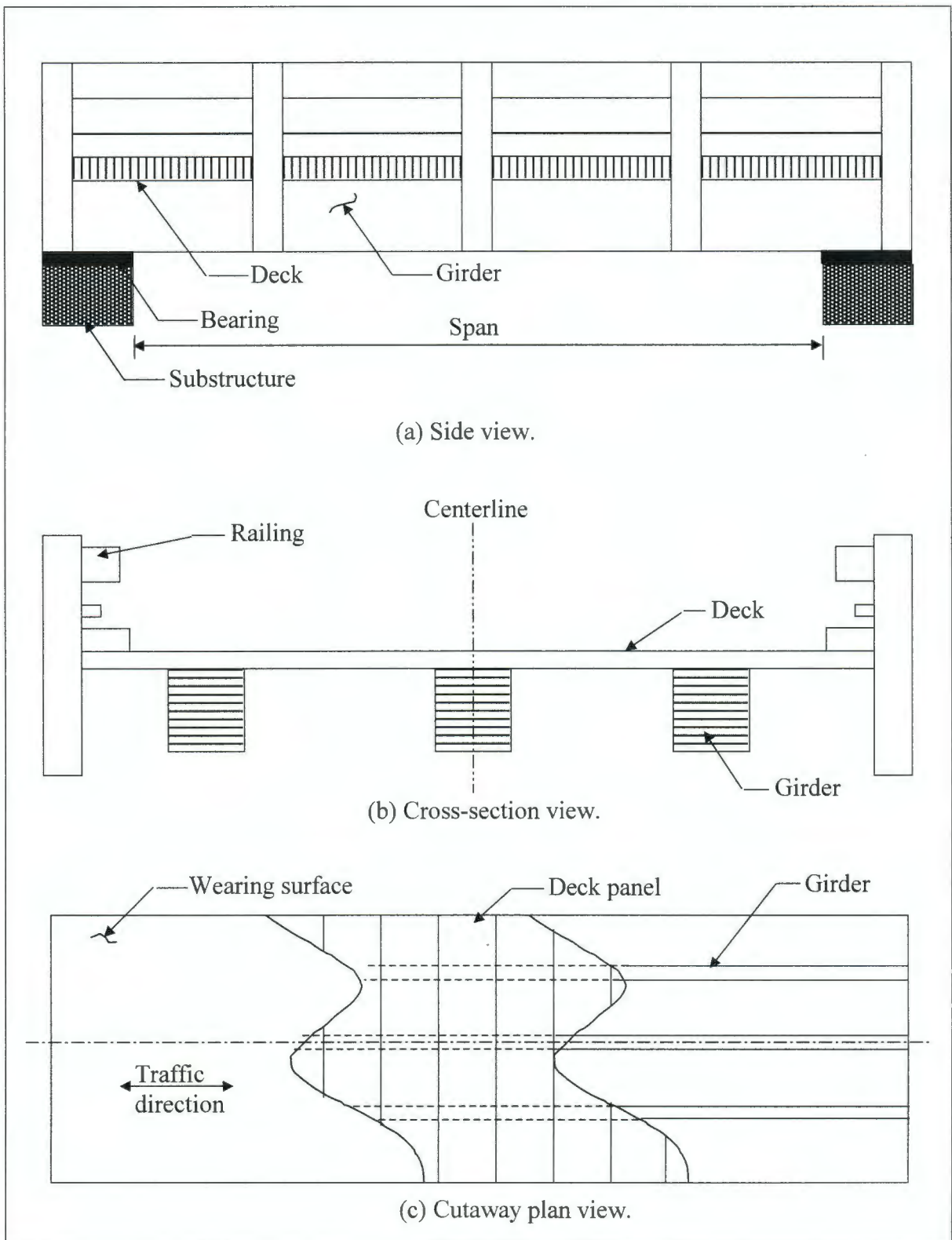
Fig. 4.2 Configuration of a typical glued laminated girder bridge.

be placed to provide a watertight deck, protecting the structure from the deteriorating effects of rain and snow.

The glulam girders are horizontally laminated members designed from the bending combinations given in Table 1 of AITC 117 – Design [14]. The combinations provide the most efficient beam section where the primary loading is perpendicular to the wide face of the laminations. The quality and strength of outer laminations are varied for different combinations to provide a wide range of tabulated design values for positive and negative bending. Glulam girders offer substantial advantages over conventional sawn lumber girders because they are manufactured in larger sizes, and can be cambered to offset dead load deflection.

4.2 Design procedures for girder bridges

Non-interconnected glulam decks are the most widely used type of glulam deck in modern timber bridge construction. They are economical, require little fabrication, and are east to install with unskilled labor and without special equipment. Since the panels are not connected to one another, each panel acts individually to resist the stresses and deflection from applied loads.

4.2.1 The non-interconnected glulam deck

The deck is assumed to act as a simple span between girders and is designed for the stresses acting in the direction of the deck span and deflection. Stresses in the direction perpendicular to the span are not critical and are not considered in design. The basic design procedures for non-interconnected glulam decks are given in detail below [1]. The sequence assumes that the panels are initially designed for bending, then checked for deflection and

shear. Deflection rather than bending stress usually controls in most applications. The designer establishes the acceptable level of deflection and this may vary for different applications.

The effective deck span, 's', is the clear distance between the supporting girders plus one-half the width of one girder, but not greater than the clear span plus the panel thickness (AASHTO 3.25.1.2). The panel width is a multiple of 1.5 inches, the net width of the individual lumber laminations. The designer should check local manufacturing and treating limitations before specifying widths over 48 inches. The deck design load is the maximum wheel load of the design vehicle. For H 20-44 and HS 20-44 loads, AASHTO special provisions for timber decks apply, and a 12,000-pound wheel load is used instead of a standard 16,000-pound wheel load. An initial estimate of the deck thickness is made based on bending and deflection considerations [1].

The wheel distribution widths in the direction of the deck span, 'b_d', and perpendicular to the deck span, 'b_t', are compliant with AASHTO regulations (AASHTO 3.25.1.1). The effective deck section, defined by a deck width, 'd_w', and thickness, 't', is designed as a beam to resist the loads and deflection produced by one wheel line of the design vehicle. Uniform dead load moment for the effective deck section can be computed as:

$$M_{DL} = w_{DL} s^2/8$$

where M_{DL} = deck dead load moment,
  w_{DL} = dead load of the deck and wearing surface over wheel load distribution width, 'b_d', and
  s = effective deck span.

When a portion of the dead load is not uniformly distributed (as when the deck supports utility lines or other components), dead load moment from these non-uniform loads is computed by assuming the deck acts as a simple span, and the moment from the additional loading is added to 'M_{DL}' computed above.

The maximum vehicle live load moment, 'M_{LL}', is computed by assuming that the deck acts as a simple span between the girders. Wheel loads are positioned laterally on the span to produce the maximum moment. When deck panels are continuous over two spans or less, bending stress is based on simple span moments. When the deck is continuous over more than two spans, the maximum bending moment is 80 percent of that computed for a simple span to account for span continuity (AASHTO 3.25.4).

Bending stress, $f_b = M/S_y$

where $M = (M_{DL} + M_{LL})$ for less than two spans, or,
          $= 0.8(M_{DL} + M_{LL})$ for more than two spans, and
          S_y = effective deck section modulus = $d_w\, t^2/6$.

For the required bending stress, a panel combination symbol is selected from AITC 117 [14]. The allowable bending stress, '$F_b{}'$', is given by the following formula:

$$F_b{}' = F_{by}\, C_F\, C_M$$

where F_{by} = tabulated bending stress from [14],
C_F = size factor of panels less than 12 inches thick [1],
C_M = moisture content factor for bending = 0.80 [1].

If $f_b \leq F_b{}'$, the initial deck thickness and combination symbol is satisfactory in bending. When 'f_b' is significantly lower than '$F_b{}'$', a thinner deck or lower grade combination symbol may be more economical; however, no changes should be made in panel thickness or combination symbol until the live load deflection is determined. If $f_b > F_b{}'$, the deck is insufficient in bending and the deck thickness or grade must be increased, or the

effective deck span must be reduced. If either of these quantities is changed, the design sequence must be repeated.

Live load deck deflection is computed by standard methods of engineering analysis, assuming the deck to be a simple span between girders. It should be noted that the modulus of elasticity must be multiplied by the moisture content factor, 'C_M', before plugging into the equation for deflection. When the deck is continuous over more than two spans, the maximum deflection is 80 percent of that computed for a simple span to account for deck continuity. Since AASHTO does not specify the acceptable live load deflection limit, the limit is left to the designer's judgment. The deck deflection is important because it directly influences the performance and serviceability of the deck, wearing surface, and mechanical connections. When deflections are large, vertical movement of the panels causes vibrations in the structure and rotation of the deck panels about the girders. This can cause bolts to loosen and asphalt wearing surfaces to crack.

Horizontal shear for dead load, 'V_{DL}', is based on the maximum vertical shear occurring at a distance from the support equal to the deck thickness, 't'. Live load vertical shear, 'V_{LL}', is computed by placing the edge of the wheel load distribution width, 'b_t', a distance 't' from the support.

Horizontal shear stress, $f_v = 1.5\ V/A_v$

where $V = V_{DL} + V_{LL}$,
         $A_v = t\ (15+2t) \leq t\ d_W$ (square inches), and
         d_W = panel width.

Allowable shear stress, $F_v' = F_{vy}\ C_M$

where F_{vy} = tabulated shear stress from [14], and
         C_M = moisture content factor for shear = 0.875 [1].

Finally, the deck overhang at exterior supports is checked using an effective span measured to the centerline of the outside girder, minus one-fourth the girder width. For vehicle live load stresses and deflection, the wheel load is positioned with the load centroid 1 foot from the face of the railing or curb. The deck stress in bending and horizontal shear must be within allowable limits as mentioned in the preceding paragraphs.

4.2.2 The glulam girder

The girders are the principal load-carrying components of the bridge superstructure. They must be proportioned to resist the applied loads and meet the serviceability requirements for deflection. The total beam system consists of beams, bearings and, in some cases, transverse bracings. Since the bridges in this report do not have transverse bracings, no further mention of these bracings will be made in this report. The basic design procedures for the glulam girders are summarized below.

At this point, the span length, 'L', measured center-to-center of bearings, the roadway width measured face-to-face of curbs (AASHTO 2.1.2), the number of traffic lanes, the number and spacing of girders, the design load vehicles, and the deck and curb configuration, are assumed to be known to the designer. An initial beam combination symbol is selected from [14]. The deck dead load supported by each girder is computed and the dead load moments are calculated assuming the girders to be simply supported at its ends. The live load moments are computed for interior and exterior girders by multiplying the maximum moment for one wheel line of the design vehicle by the applicable moment distribution factors given in [1].

The initial girder size is estimated based on the deck dead load moment and vehicle live load moment, assuming the size factor, 'C_F', controls allowable bending stress. The girder dead load moment is unknown at this point.

$$S_x\, C_F = M/F_b{}'$$

where $S_x\, C_F$ = required girder section modulus adjusted by size factor,
         M = applied dead load and live load bending moment,
         $F_b{}'$ = allowable bending stress = $F_{bx}\, C_M$,
         F_{bx} = Tabulated bending stress value from [14] for particular combination symbol, and
         C_M = moisture content factor for bending = 0.80 [1].

An interactive chart in [1] allows the designer to obtain the required girder depth for the selected '$S_x\, C_F$' value. Girder design usually favors a relatively narrow, deep section with a depth to width of ratio of 4:1 and 6:1. The beam dead load moment is computed for the chosen beam size and added to the deck load and live load moments. A revised beam size is selected and this interactive process is continued until a satisfactory girder section is obtained. Applied bending stress is then computed for the member:

$$f_b = M/S_x \text{ not greater than } F_b{}' = F_{bx}\, C_M\, C_F$$

Girder size based on bending stress must be checked for lateral stability. If stability controls over size factor, it is generally more economical to reduce unsupported girder length by providing bracings. If this is not practical, the girder must be redesigned for the lower stress required for stability.

Live load deflections are computed from standard methods of engineering analysis. The distribution of deflection to bridge girders depends on the transverse deck stiffness. On single lane bridges with glulam decks, the deflection produced by one vehicle (two wheel lines) is assumed to be resisted equally by all the girders. On multi-lane bridges, deflection

can be distributed using the distribution factor (DF) for girder moment [1], or by assuming

that all girders equally resist the deflection produced by the simultaneous loading of one

vehicle in each traffic lane. Criteria for maximum deflection are based on designer judgment,

but should not exceed $L/360$.

Dead load horizontal shear, 'V_{DL}', is based on the maximum vertical shear occurring a

distance from the support equal to the girder depth, 'd'. Live load vertical shear, 'V_{LL}', is

computed at the lesser distance of '$3d$' or $L/4$. Applied horizontal shear stress, 'f_v', must not

be greater than the allowable stress, 'F_v'', as given by:

$$f_v = 1.5 \ V/A \ \leq \ F_v' = F_{vx} \ C_M$$

where $V = V_{DL} + V_{LL}$,
          A = girder cross-section area,
          F_{vx} = tabulated shear stress value from [14], and
          C_M = moisture content factor for shear = 0.875 [1].

When $f_v \leq F_v'$, the girder is adequately proportioned for horizontal shear. If $f_v > F_v'$,

the girder is insufficient in shear and the cross-sectional area must be increased. Bearing area

at girder reactions must be sufficient to limit stress to an allowable level. The total dead load

reaction, 'R_{DL}', at each girder is computed. The live load reaction, 'R_{LL}', at each girder is

computed by multiplying the maximum reaction for one wheel line by the applicable

distribution factor for reactions [1].

Required bearing length = $(R_{DL} + R_{LL})/(b \ F_{c\perp}')$
Applied bearing stress = $(R_{DL} + R_{LL})/A \ \leq \ F_{c\perp}'$
where $F_{c\perp}'$ = Allowable compressive stress perpendicular to the grain = $F_{c\perp x} \ C_M$,
          $F_{c\perp x}$ = Tabulated compressive stress perpendicular to the grain [14], and
          C_M = Moisture content factor for bearing = 0.53 [1].
          A = Bearing area.

The camber is based on span length and girder configuration. For girders with spans

greater than 50 feet, camber is generally 1.5 to 2.0 times the computed dead load deflection.

For spans less than 50 feet, camber is 1.5 to 2.0 times the dead load deflection plus one-half the vehicle live load deflection. Camber for single beams is specified as a vertical offset at the girder centerline. On multiple-span continuous girders, camber may vary along the girder and should be specified at the center of each span segment. Finally, the bearing shoe, bearing pad, girder to deck attachment bolts, and anchorage system is designed.

4.3 Analytical model of bridge

The ANSYS software [8] was used to describe the bridge behavior analytically because of its vast element library and powerful analysis techniques. The model was assembled by modeling the deck panels, girders, and curbs, if present. The panels and girders were modeled using quadrilateral, elastic, and orthotropic shell elements. The curbs were modeled with 3-D elastic beam elements connected to the deck by rigid links. All these elements are present in the ANSYS element library.

4.3.1 Modeling the deck panels

The deck panels are laid out transverse to the girders. The deck panels are non-interconnected and the assumption made in this regard is that the asphalt-wearing surface and friction between the adjacent panels contribute insignificantly towards continuity and load distribution between the panels.

The four-node shell element (SHELL63) [11] was chosen to model the deck panel. This element has already been described in Sec. 2.3.1. This element can be used to model the orthotropic properties of wood and is defined by thickness, longitudinal and transverse moduli of elasticity, shear modulus and major or minor Poisson's ratio. Fig. 2.2 shows the SHELL63 element used to model the deck panels.

4.3.2 Modeling the girders

The girders were also modeled with SHELL63 (Fig. 2.2) elements. The depth of the girder was divided into shell elements and the thickness of the shell elements was the width of the girder. Nodes were located at the bottom of the girders for provision of supports. Bhari's research [5] showed that the girder and deck could be assumed to be fully composite with each other. Hence, in the model, the full composite action was idealized by making the connecting nodes between the deck panels and the girders common nodes.

4.3.3 Modeling the curbs

The curbs were modeled using 3-D beam (BEAM4) (Fig. 2.3) elements. This two-node element has already been discussed in Sec. 2.3.2 and Sec. 2.3.4. The preprocessor gives the user the option to include curbs in the model. This option becomes very useful when a parametric study of the behavior of curbs on the bridge needs to be done.

It should be noted that the dead load or the permanent weight of all the structural and non-structural components of the bridges, including the roadway, sidewalks, railing, and wearing surface, were not included in the load. Their contribution to the behavior of the girder bridge was assumed to be insignificant.

4.3.4 Modeling the loads and abutment supports

The bridge was assumed to be simply supported since this assumption would be closer to the real situation and deflection and stress results would be conservative. The live load applied was truck wheel loads. The dead loads of the deck, the wearing surface, curbs, and the railings were neglected. Field data were measured under the effects of live loads only. The concentrated load (user input) is applied as a pressure load on the element upon

which the load falls with a pressure value equal to the value of the concentrated load divided by the area of the element. In a situation where the concentrated load fell on a node of an element, the load was applied as a pressure on the closest surrounding element (chosen by smaller element number).

4.4 Preprocessor and postprocessor for glulam girder bridges

These programs were developed using ANSYS Parametric Design Language (APDL) that is available inside the ANSYS software. The purpose of these programs was to reduce user input and provided required results in a more presentable format. The inputs required for the programs are: 1) basic dimensions of the bridge, 2) material properties of the deck, girders, and curbs (if provided), 3) position and magnitude of the loads. The user is also given an option to choose an element size to mesh the deck and the girders. This option was exercised in a mesh sensitivity study for the three case studies analyzed in the next chapter.

These programs store the required output in a tabular format in a filed called 'Results.dat', that is created in the home directory of the user. This output can then be imported into an Excel spreadsheet and the user can obtain a graphical representation of the results. The programs are capable of analyzing only single spans and the results extracted are along the midspan of the bridge (along the skew centerline for the skew bridges). The output file contains midspan girder deflections and midspan girder stresses. A listing of the program with comments is provided in Appendix B. A complete 'User Manual' to the program with an example is provided in Appendix C. Fig. 4.3 shows the analytical model of a typical glulam girder bridge.

(a) Model of a deck panel.

(b) Model of a girder.

(c) Model of girder bridge.
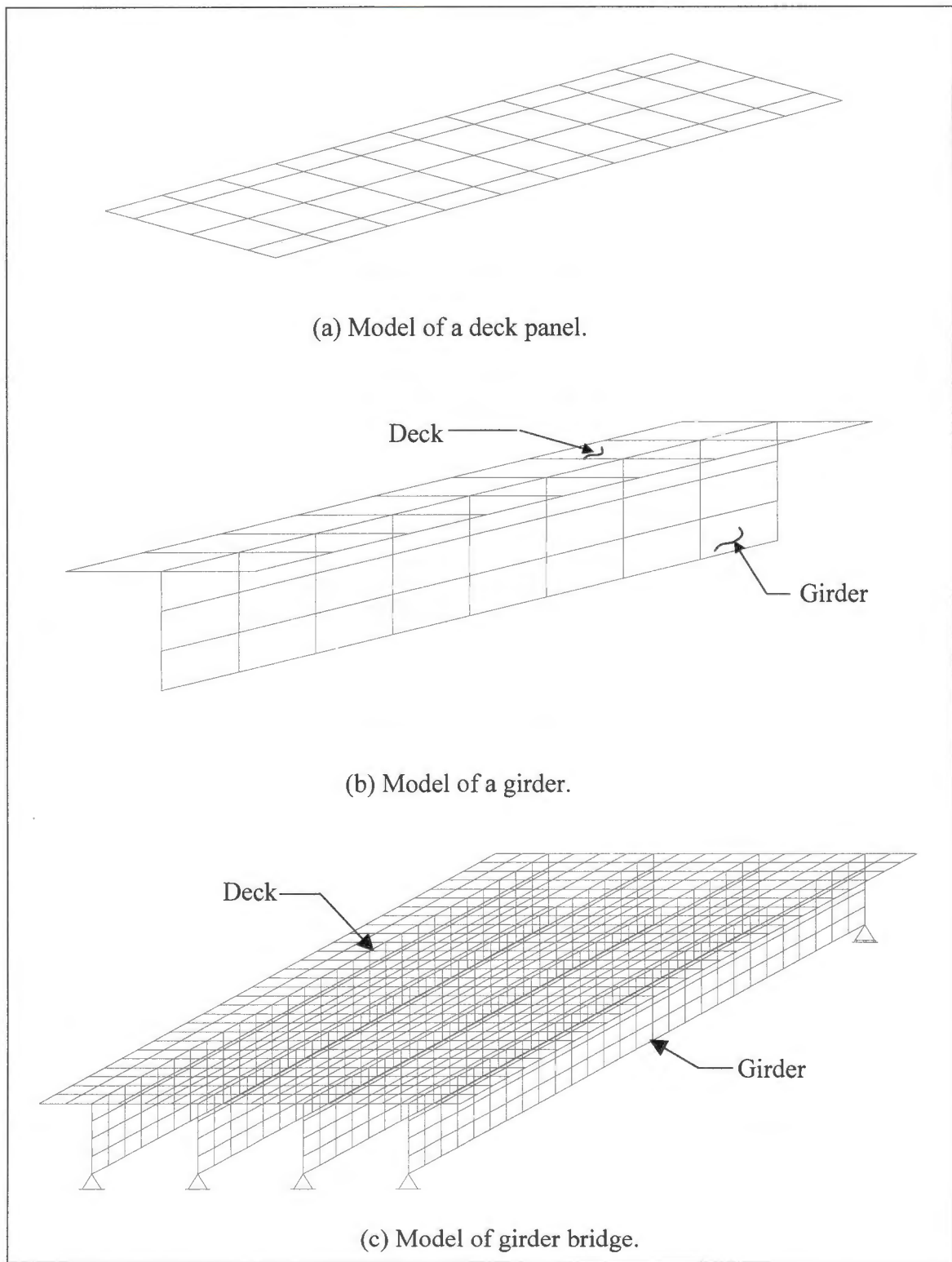
Fig. 4.3 Analytical model of a typical girder bridge with four girders.

5. COMPARISON OF EXPERIMENTAL AND ANALYTICAL DATA FOR GIRDER BRIDGES

5.1 General

Four case studies were undertaken to test the model developed. The case studies were the Cow Gulch Bridge in Montana, the Wittson Bridge in Alabama, the Chambers County Bridge in Auburn, and the Hibbard Creek Bridge in Montana. The first three bridges are right-angled bridges and the fourth one is a skew bridge with a skew of 30 degrees, left end ahead.

A parametric study was also performed for each bridge to determine the sensitivity of the results. This study was done by varying different input parameters such as material properties of the deck and girders, end restraint, and curb height. The above were first performed on the Cow Gulch Bridge in Montana. Based on this study, the parametric study on the other bridges was done only for parameters that had a significant effect on the results.

5.2 The Cow Gulch Bridge, Montana

5.2.1 Description

The Cow Gulch Bridge (Fig. 5.1) is owned by Yellowstone County in Montana and was built from a grant received by the county from the *'Wood in Transportation Program'* in 1996 . The focus of the grant was to construct economical timber bridges, and to encourage involvement by a local timber laminating facility. The bridge is made of Coast

(a) Elevation view of the Cow Gulch Bridge.

(b) End view of the Cow Gulch Bridge showing the railing system.

Fig. 5.1 The Cow Gulch Bridge in Montana.

Douglas Fir Grade 2, with six girders supporting the deck panels. The deck panels are about 4 feet wide, 28 feet long, and 5.125 inches thick and are laid transversely on the girders. They are connected to the girders by lag bolts at 6 inches in from each edge of the panel. The bridge measured 40 feet in span, measured center-to-center of bearings. The girders have a nominal width of 8.75 inches, and a nominal thickness of 28.5 inches. The curbs on the bridge have a cross-section dimension of 8 inches by 8 inches. The material properties for the analytical model were obtained from the Douglas Fir Handbook [9] for the particular grade of lamination used on bridge. The deck panels have a longitudinal and transverse modulus of elasticity of 1800 kips-per-square inch (ksi) and 130 ksi respectively. The shear modulus of the deck panels is about 100 ksi. The girders have a longitudinal and transverse modulus of elasticity of 2000 ksi and 240 ksi respectively. The shear modulus of the girders is about 106 ksi. Refer to Fig. 5.2 for the design configuration of the bridge.
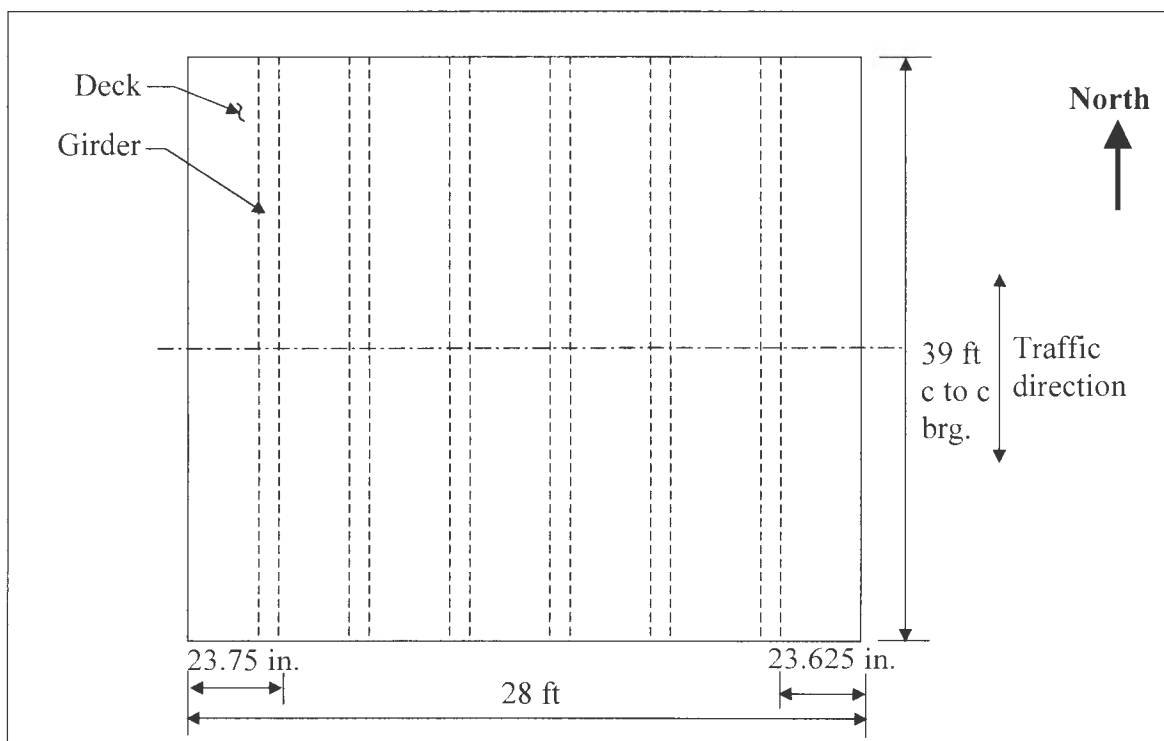
Fig. 5.2 Plan layout of the Cow Gulch Bridge.

5.2.2 Loading

Six load tests were done on this bridge. The load test vehicles consisted of two fully loaded three-axle dump trucks with gross vehicle weights of 50,920 pounds (Truck 1) and 50,080 pounds (Truck 2). The rear axles on Truck 1 weighed about 36,820 pounds while the rear axles on Truck 2 weighed about 36,000 pounds. The rear axles weighed about 75% of the respective gross vehicle weights. The vehicles were positioned longitudinally on the bridge so that the two rear axles were centered at midspan of the bridge. The transverse vehicle track width, measured center-to-center of the rear tires, was 6 feet. For Load Case 1, Truck 1 was placed on the east lane of the bridge, 2 feet away from the longitudinal centerline of the bridge. For Load Case 2, Truck 2 was placed on the west lane of the bridge, 2 feet away from the longitudinal centerline of the bridge. For Load Case 4, Truck 1 was placed 28 inches from the east edge of the bridge while for Load Case 5, Truck 2 was placed 28 inches from the west edge of the bridge. Load Case 3 was a combination of Load Cases 1 and 2, while Load Case 6 was a combination of Load Cases 4 and 5. Fig. 5.3 shows the configuration of the trucks and the load positions on the bridge.

5.2.3 Parametric study

A parametric study on the Cow Gulch Bridge was done for Load Cases 1 and 3. The input data described in Section 5.2.1 was used to make an initial run on the program. The longitudinal modulus of elasticity of the girders was then varied and a comparative plot for girder deflections at midspan of bridge was made. Comparison was also done with respect to the experimental data from field tests. Similar deflection comparisons were made by
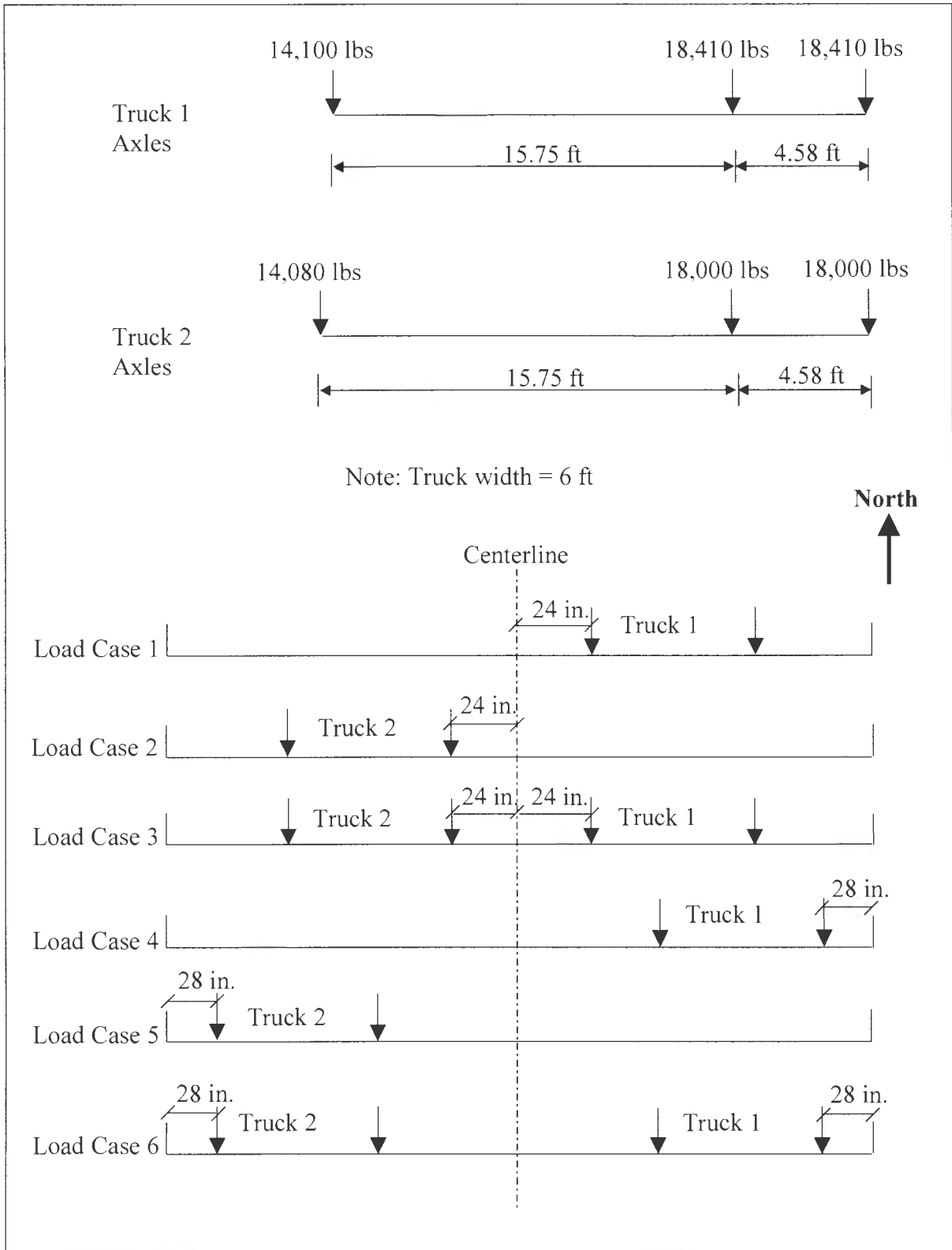
Fig. 5.3 Configuration of trucks and load positions for various load cases.

individually changing the height of the curbs, material properties of the deck, and end restraint. Figs. 5.4 to 5.12 show the comparative plots for the full parametric study. The analytical model of Cow Gulch Bridge is shown in Fig. 5.49. A comparative study of experimental values and analytical results for midspan girder deflections was also done for all the other load cases. The deflection plots for this comparison are shown in Figs. 5.13 to 5.16. A mesh sensitivity study was also done on the analytical model of the bridge to determine the adequate size of the element to be used. The element size should be small enough to yield reasonable results yet large enough to minimize computer time.

5.2.4 Comparison of maximum design stress and maximum stress from analysis

The maximum design stress for the girders on the Cow Gulch Bridge was computed using the procedure described in Section 4.2.2. The maximum girder bending stress at midspan obtained using the program was 1,250 psi. The maximum design stress was equal to 1,560 psi and was calculated with the following input:

DF = 0.96 wheel-lines per girder	Width of girder = 8.75 in
M_{WL} = 1926 in-kips	Depth of girder = 28.5 in
M_{LL} = 1849 in-kips	S_x = 1185 in^3
M_{DL} = 0 in-kips	f_b = 1.56 ksi = 1,560 psi

From the stress comparison, it can be concluded that the finite element result compares well with the design stress computed from the design manual [1].

5.2.5 Results and discussion

Figs. 5.4 to 5.12 illustrate the results from the parametric study performed on the Cow Gulch Bridge for Load Cases 1 and 3. Upon varying the longitudinal modulus of elasticity of the girders from 1800 ksi to 2200 ksi, maximum deflections in the girders reduced by about

13% and maximum stresses in the girders increased by about 5% for both Load Cases 1 and 3 (Figs. 5.4 and 5.5). Thus, longitudinal modulus of elasticity may be a significant parameter influencing the deflection curve of the bridge. Deflection and stress curves also showed a significant change near the edges of the bridge when the curb dimensions were varied (Figs. 5.6 and 5.7). However, the changes towards the center of the bridge were minimal; thus, we can conclude that the effect of curbs is restricted to regions near the edge of the bridge. Upon varying curb height from 0 to 8 inches, maximum deflections and stresses in the exterior girders decreased by 10% for both load cases.

Differences in maximum deflections and stresses in the girders were less than 5% when the longitudinal and transverse modulus of elasticity of the deck was altered (Figs. 5.8 to 5.11). These differences may be assumed to be negligible and hence, it can be safely assumed that variation in deck material properties do not affect bridge performance significantly. When the supports of the bridge were changed from simple supports to fixed supports, the difference between the deflection curves was significant (Fig. 5.12). In reality, the supports usually act between the simply supported and fixed end conditions. Since the experimental deflections, in general, were lesser than the analytical results, this may be due to some effect from the rotational restraints, if any, at the abutments or/and due to the assumed material properties in the model. Deflection curves for all the other load cases show that the analytical results are comparable with the experimental observations (Figs. 5.13 to 5.16). A mesh sensitivity analysis for the Cow Gulch Bridge (using Load Case 1 as the load for the sensitivity study) indicated that when the element size was 12 inches by 12 inches, the deflection and stress results reached convergence (Fig. 5.17). The aspect ratio was kept close to unity for the mesh sensitivity study.

74
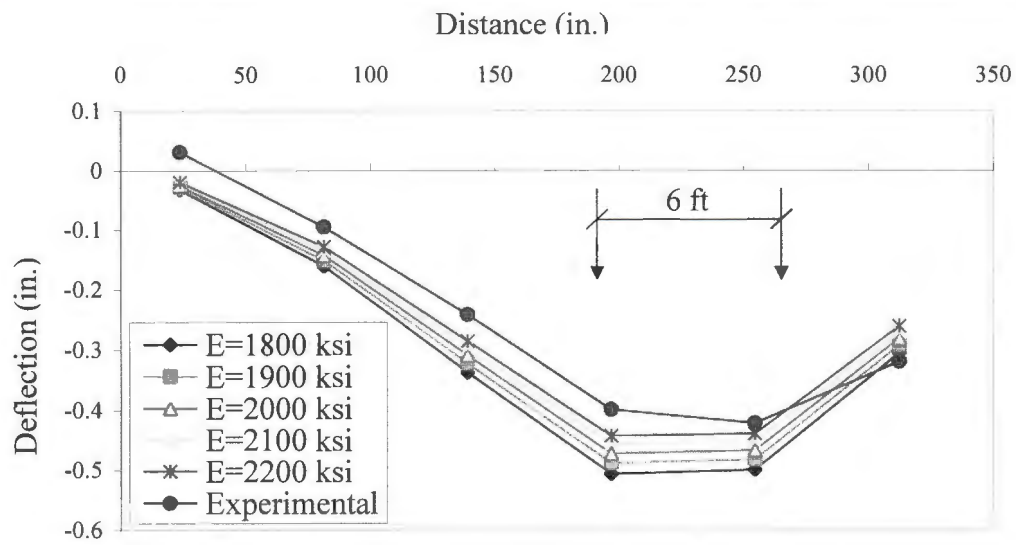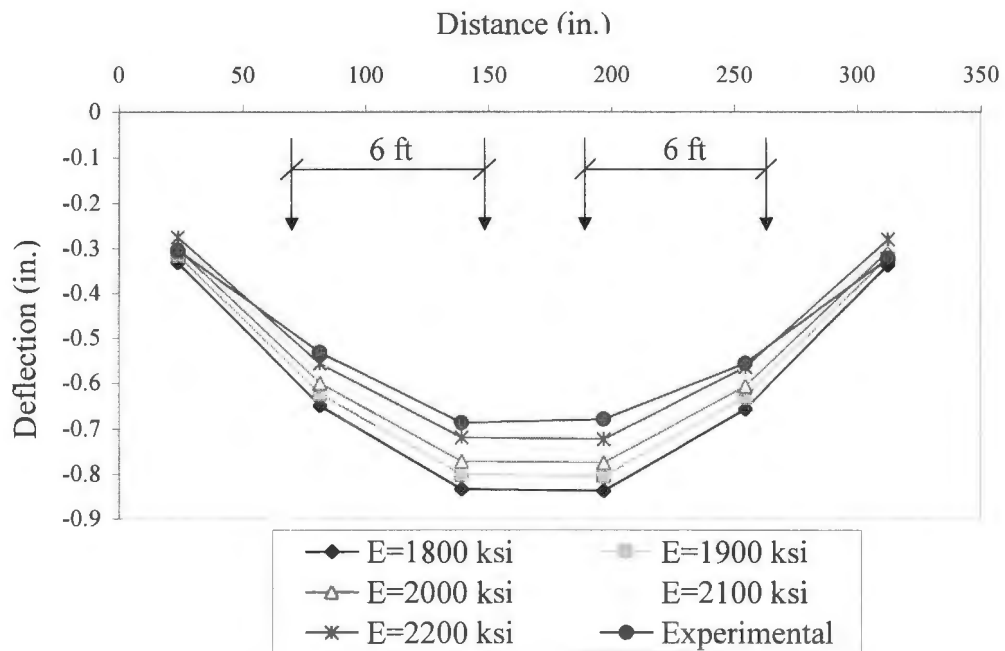
(a) Load Case 1.

(b) Load Case 3.

Fig. 5.4 Comparison of girder deflections at midspan upon changing longitudinal modulus of elasticity of the girders for Load Cases 1 and 3.
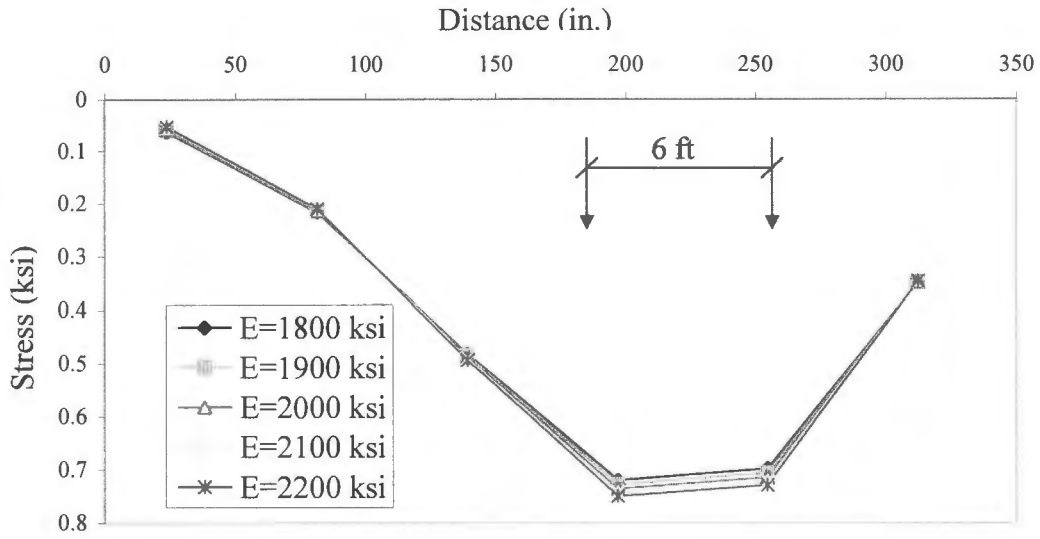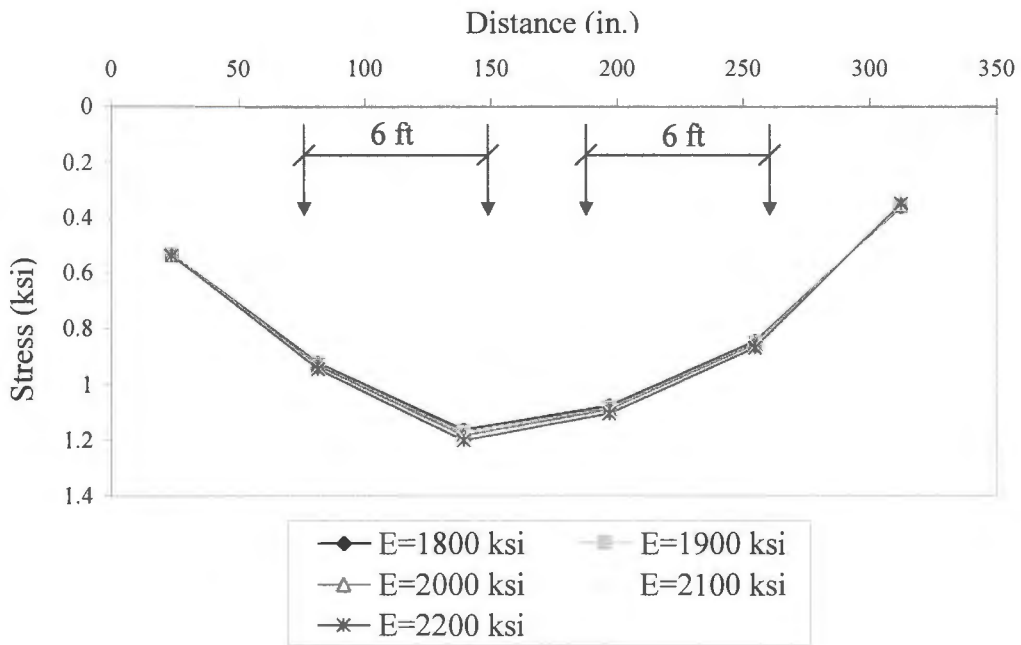
(a) Load Case 1.

(b) Load Case 3.

Fig. 5.5 Comparison of analytical girder bending stresses at midspan upon changing longitudinal modulus of elasticity of the girders for Load Cases 1 and 3.
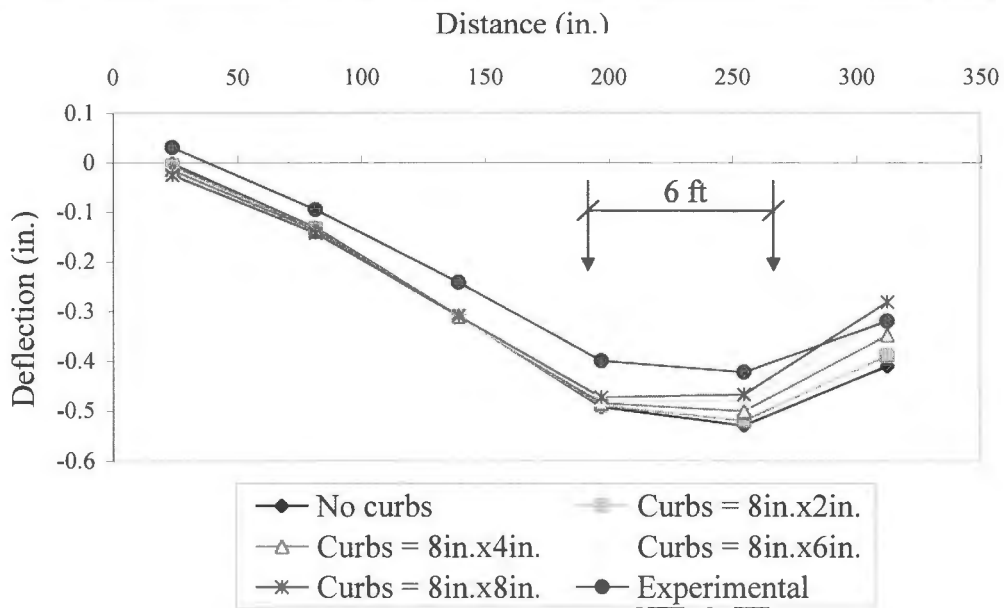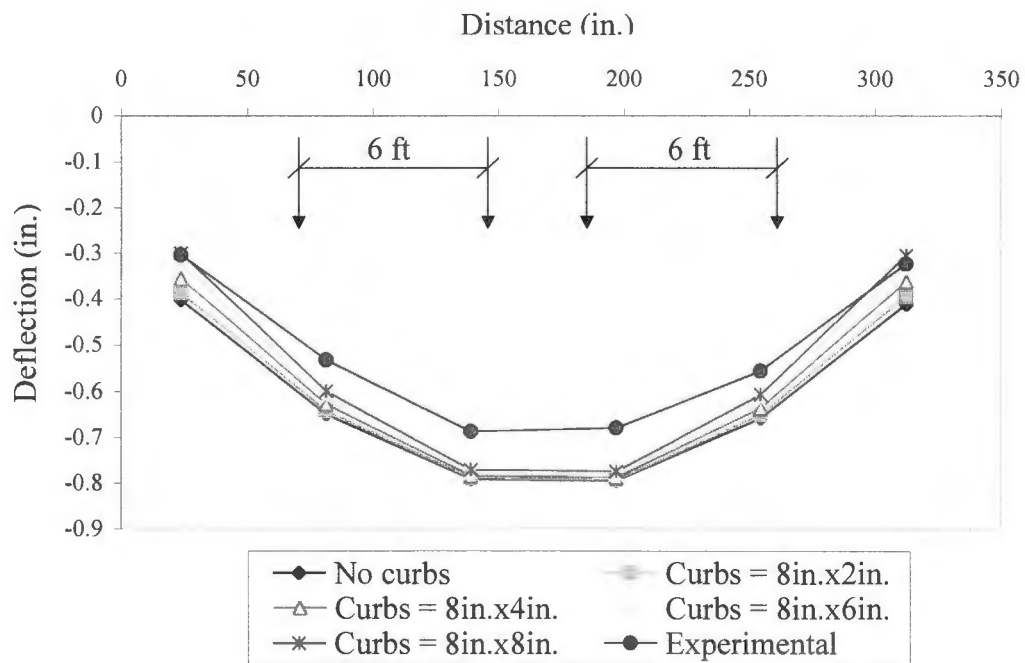
(a) Load Case 1.

(b) Load Case 3.

Fig. 5.6 Comparison of girder deflections at midspan upon varying curb height for Load Cases 1 and 3.

(a) Load Case 1.

(b) Load Case 3.

Fig. 5.7 Comparison of analytical girder bending stresses at midspan upon varying curb height for Load Cases 1 and 3.
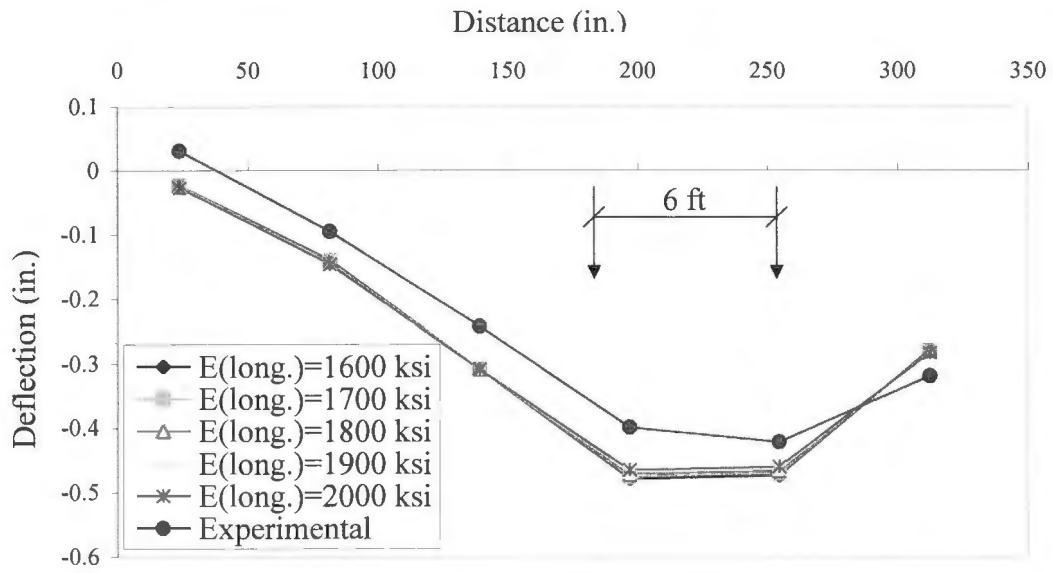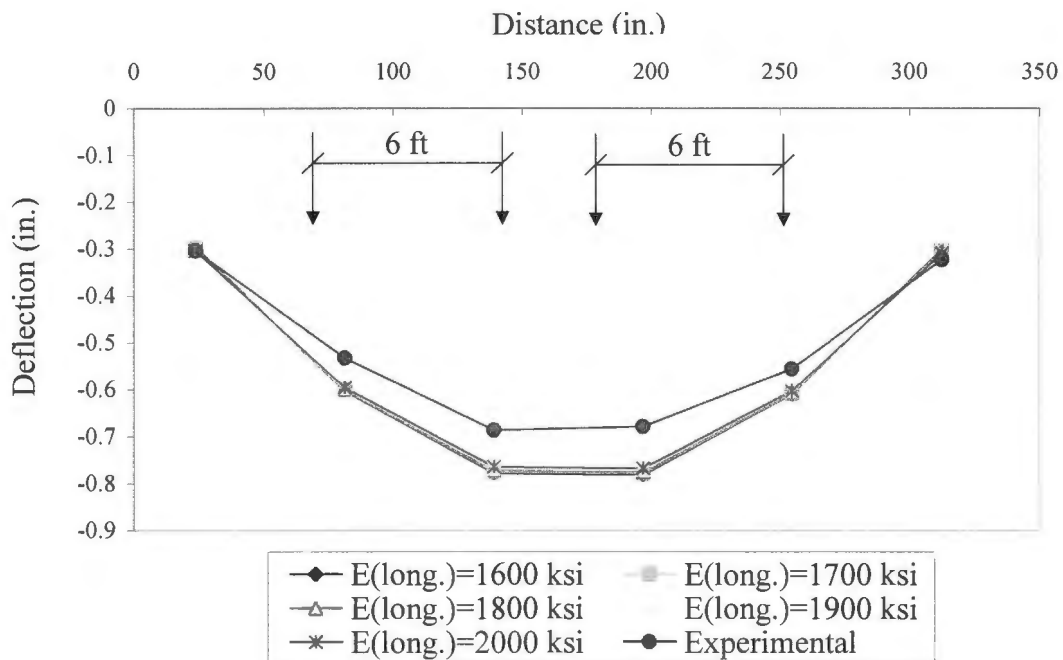
(a) Load Case 1.

(b) Load Case 3.

Fig. 5.8 Comparison of girder deflections at midspan upon changing longitudinal modulus of elasticity of the deck for Load Cases 1 and 3.
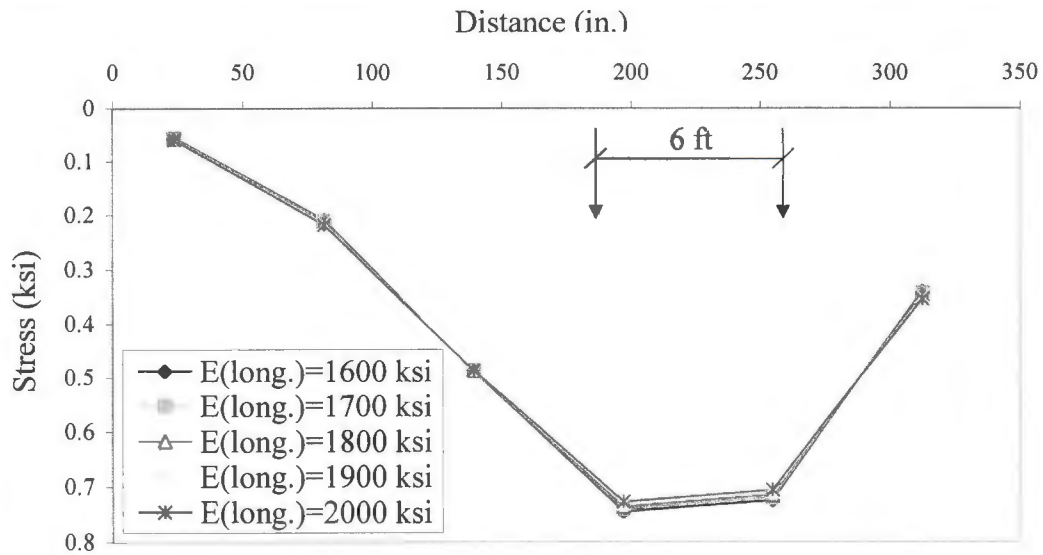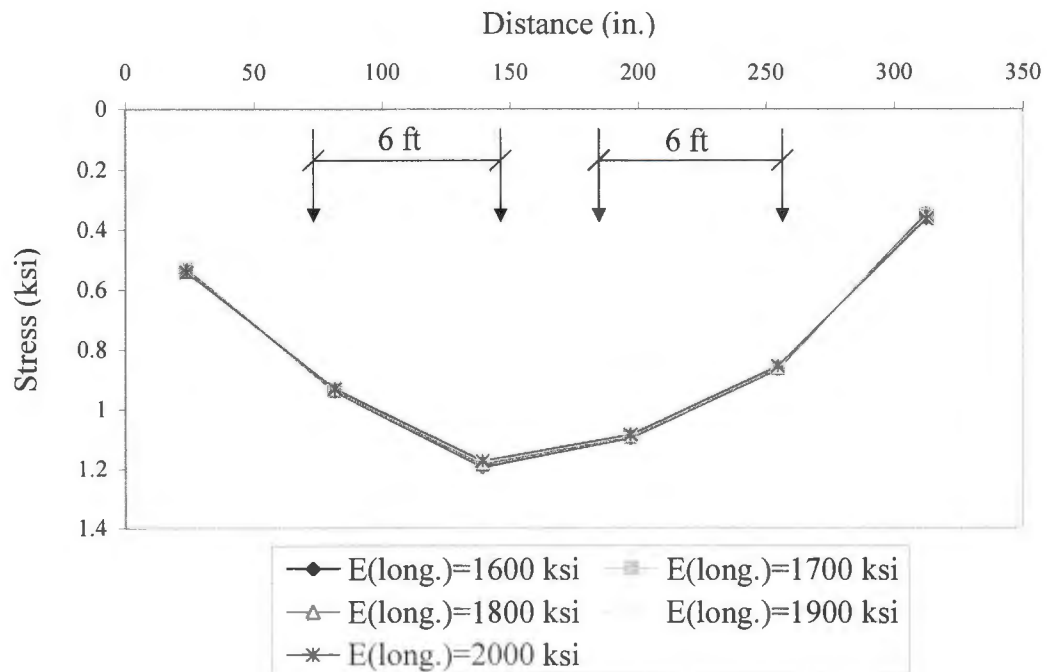
(a) Load Case 1.

(b) Load Case 3.

Fig. 5.9 Comparison of analytical girder bending stresses at midspan upon changing longitudinal modulus of elasticity of the deck for Load Cases 1 and 3.
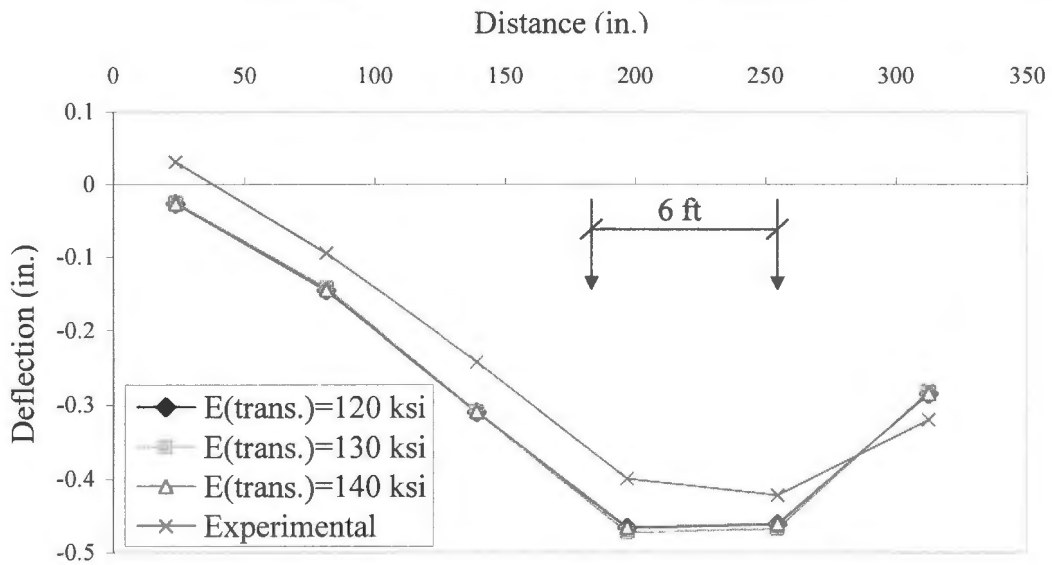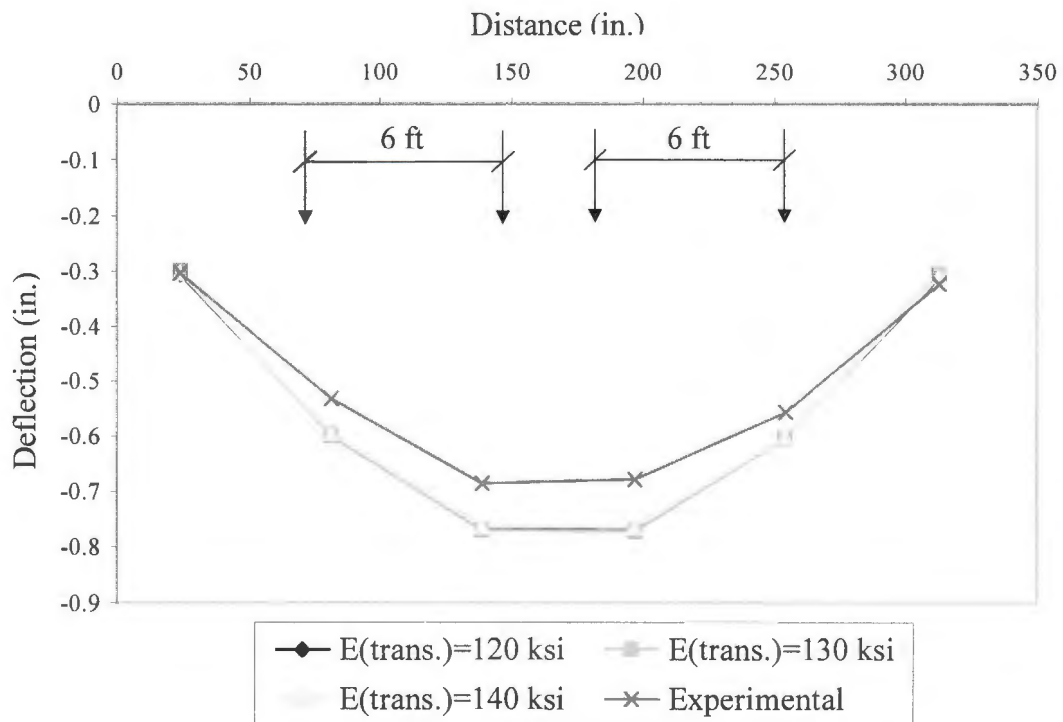
(a) Load Case 1.

(b) Load Case 3.

Fig. 5.10 Comparison of girder deflections at midspan upon changing transverse modulus of elasticity of the deck for Load Cases 1 and 3.
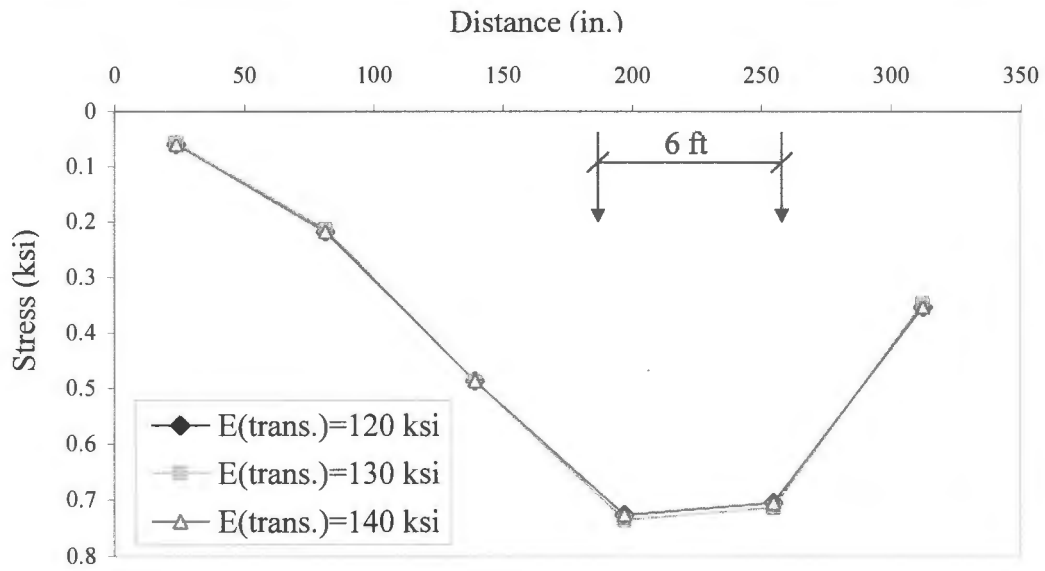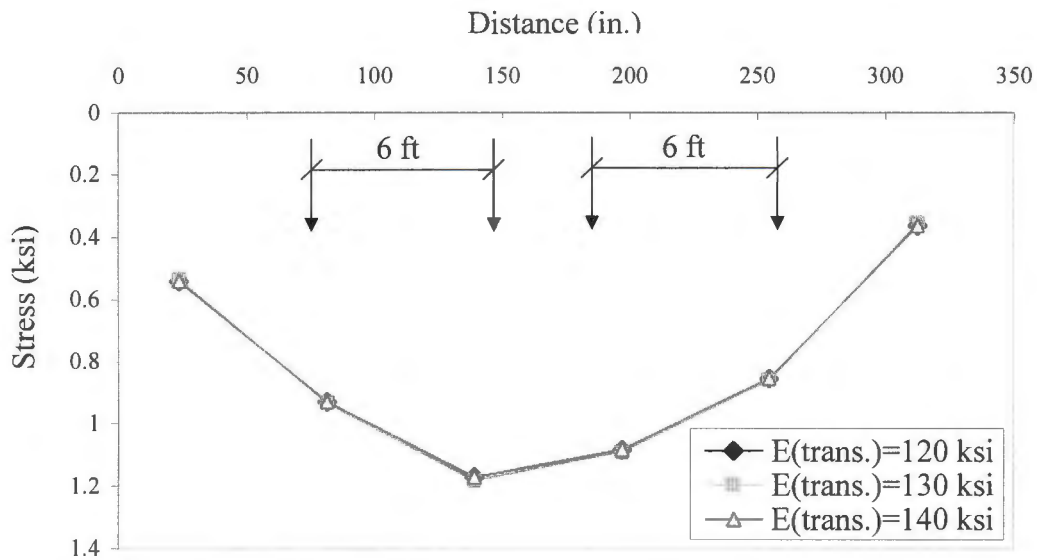
(a) Load Case 1.

(b) Load Case 3.

Fig. 5.11 Comparison of analytical girder bending stresses at midspan upon changing transverse modulus of elasticity of the deck for Load Cases 1 and 3.
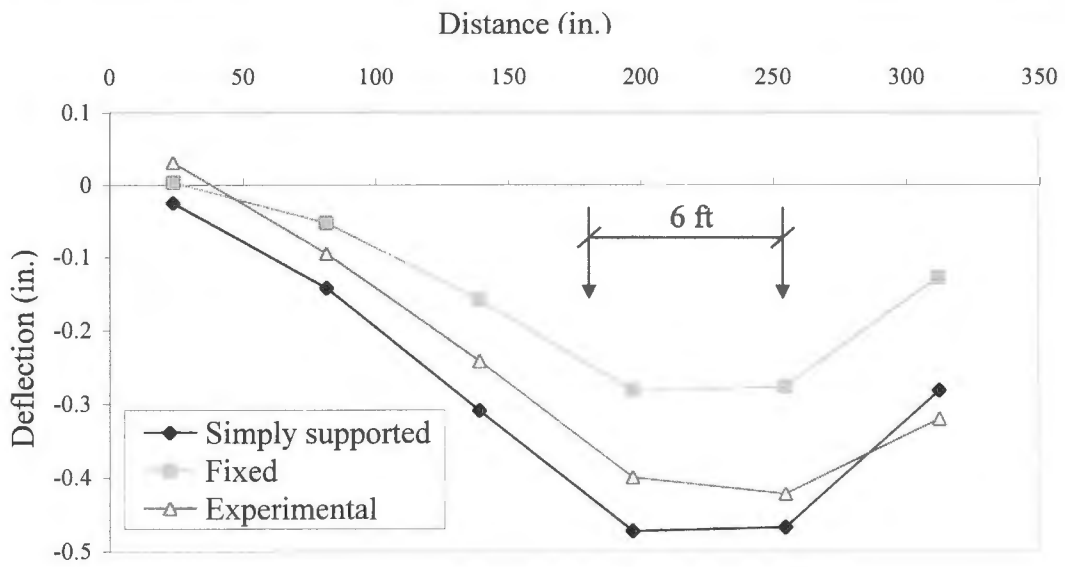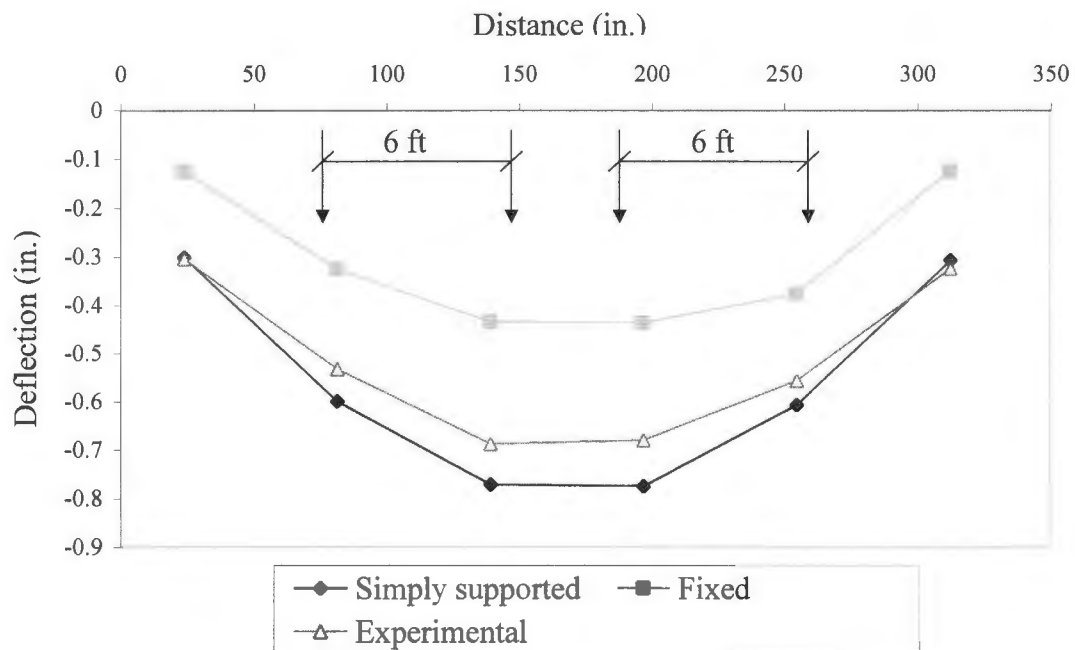
(a) Load Case 1.

(b) Load Case 3.

Fig. 5.12 Comparison of girder deflections at midspan upon changing support conditions for Load Cases 1 and 3.
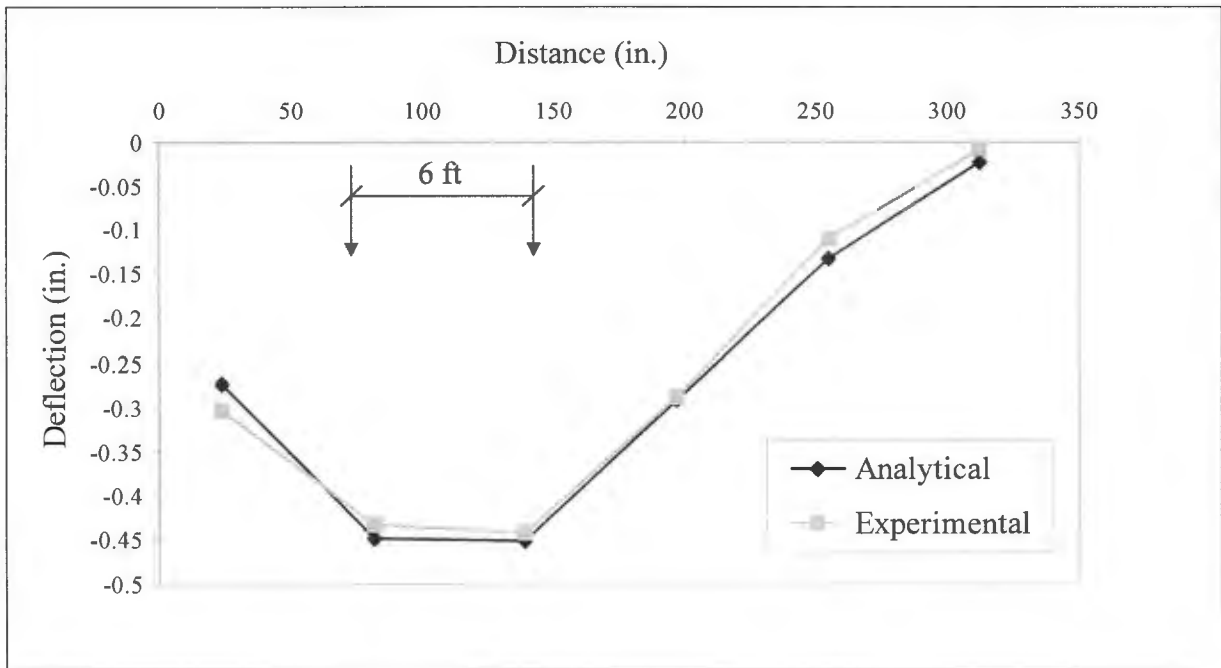
Fig. 5.13 Comparison of girder deflections at midspan for Load Case 2.
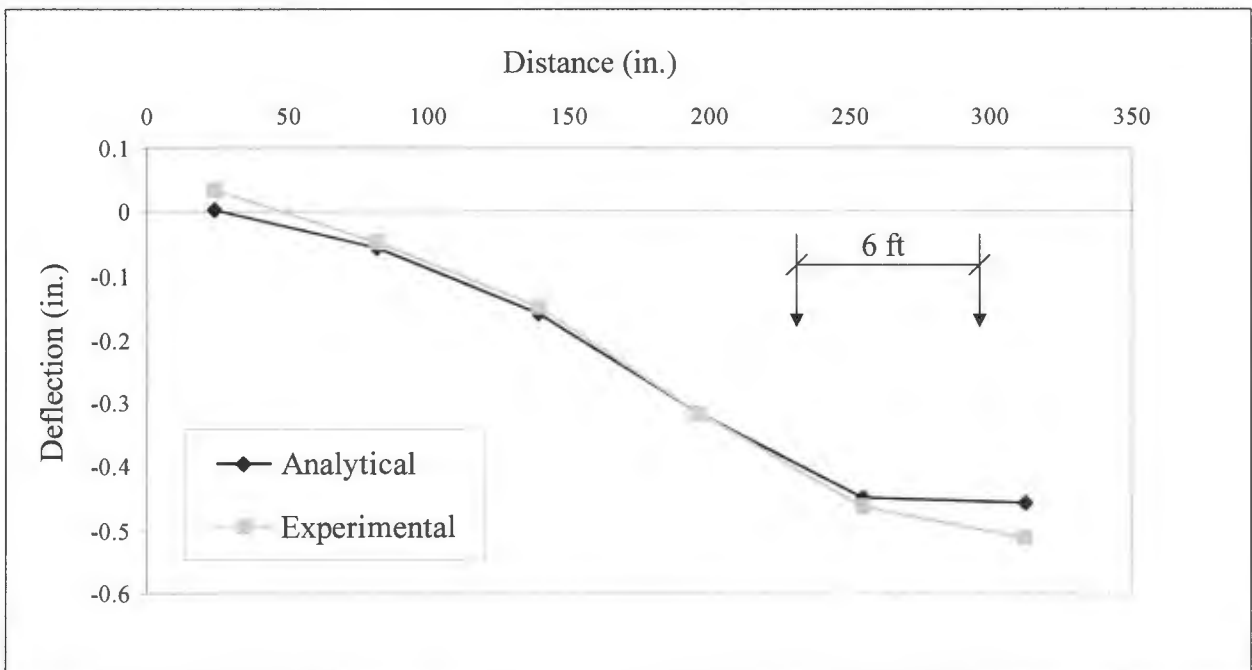
Fig. 5.14 Comparison of girder deflections at midspan for Load Case 4.
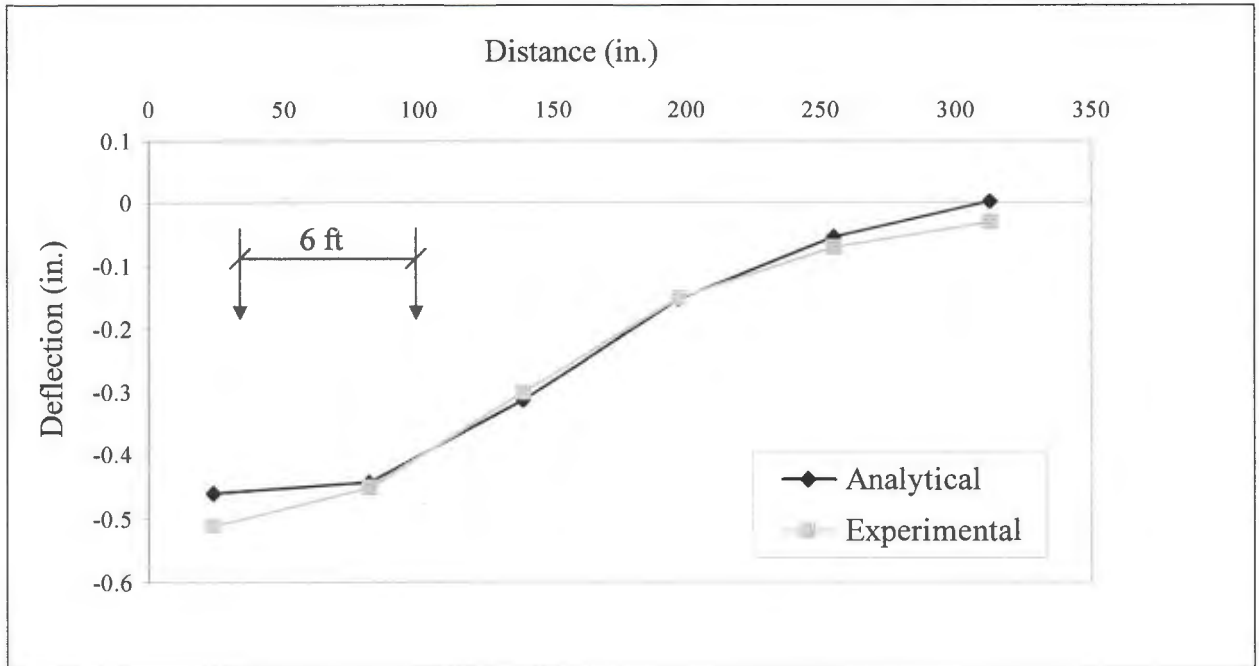
Fig. 5.15 Comparison of girder deflections at midspan for Load Case 5.
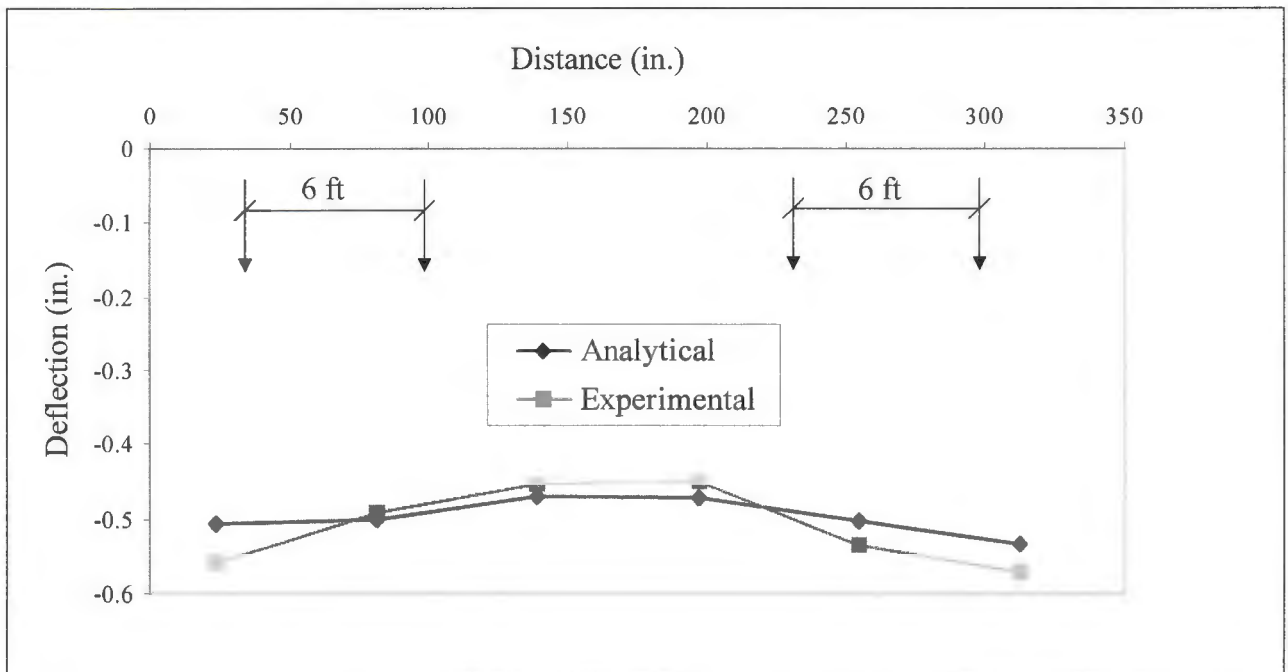
Fig. 5.16 Comparison of girder deflections at midspan for Load Case 6.
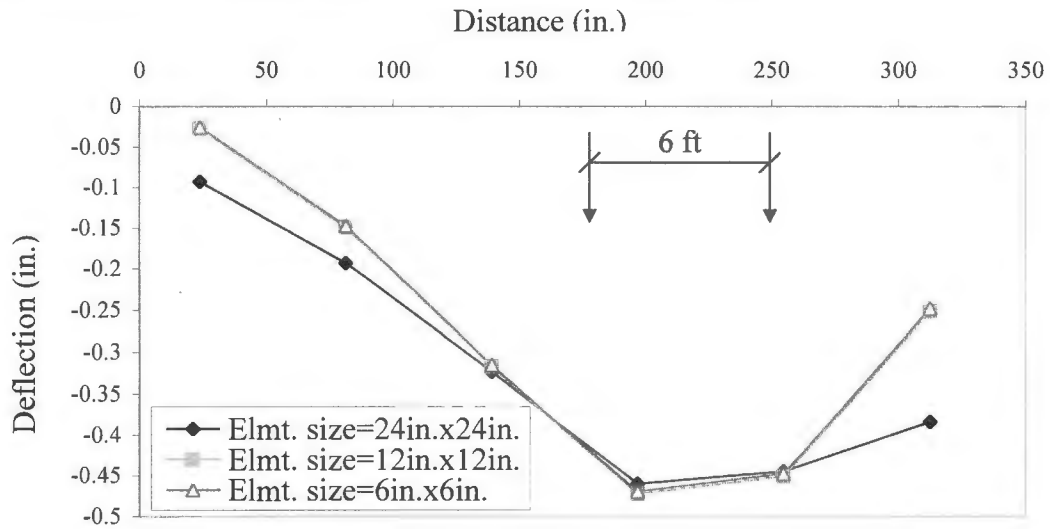
(a) Deflection curves at midspan.
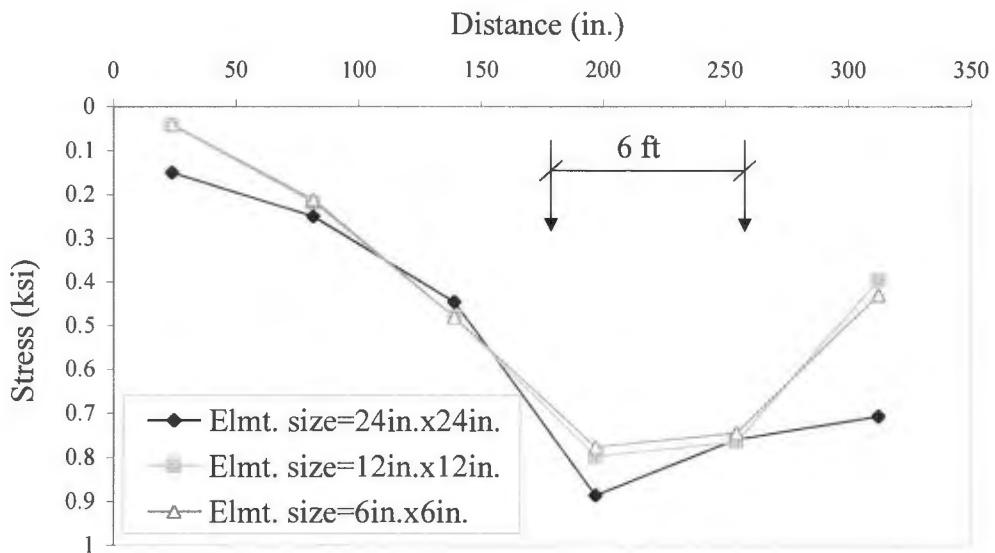
(b) Bending stress curves at midspan.

Fig. 5.17 Comparison of analytical girder deflections and bending stresses at midspan for mesh sensitivity analysis of the Cow Gulch Bridge.

5.3 The Wittson Bridge – Span 3, Alabama

5.3 Description

The Wittson Bridge is a single lane bridge consisting of four simple spans: 51.3 feet at Span 1, 51.3 feet at Span 2, 102 feet at Span 3, and 35 feet at Span 4 (all measurements made from center-to-center of bearings). A photo of the bridge appears in Fig. 5.18. Although the ends of the stringers of adjacent spans were separated by a 1.5 to 3 inch gap, the deck panels overlapped from one span to another, creating possible rotational continuity. The first and third spans (Span 1 and Span 3) were tested for midspan girder deflections using static trucks as live load cases. This report, however, deals with only Span 3 of the bridge. The bridge is made of Southern Yellow Pine with four girders spaced at 51 inches on center, supporting the deck. The deck panels are about 4 feet wide, 15 feet long, and 5.125 inches thick. These panels are connected to the girders by lag screws at 6 inches in from each edge of the panel.

The girders have a nominal width of 10.625 inches and a nominal thickness of 63.25 inches. Steel guardrail on timber posts are installed on both sides of the bridge. The material properties for the analytical model were obtained from the AITC-117 Design Manual [9] for the particular grade of lamination used on bridge. The deck panels have a longitudinal and transverse modulus of elasticity of 1930 ksi and 240 ksi, respectively. The shear modulus of the deck panels is about 106 ksi. The girders have a longitudinal and transverse modulus of elasticity of 1930 ksi and 240 ksi, respectively. The shear modulus of the girders is about 106 ksi. Refer to Fig. 5.19 for the design configuration of the bridge.

(a) Elevation view of the Wittson Bridge.

(b) End view of the Wittson Bridge showing the girders.

Fig. 5.18 The Wittson (Tuscaloosa) Bridge in Alabama.
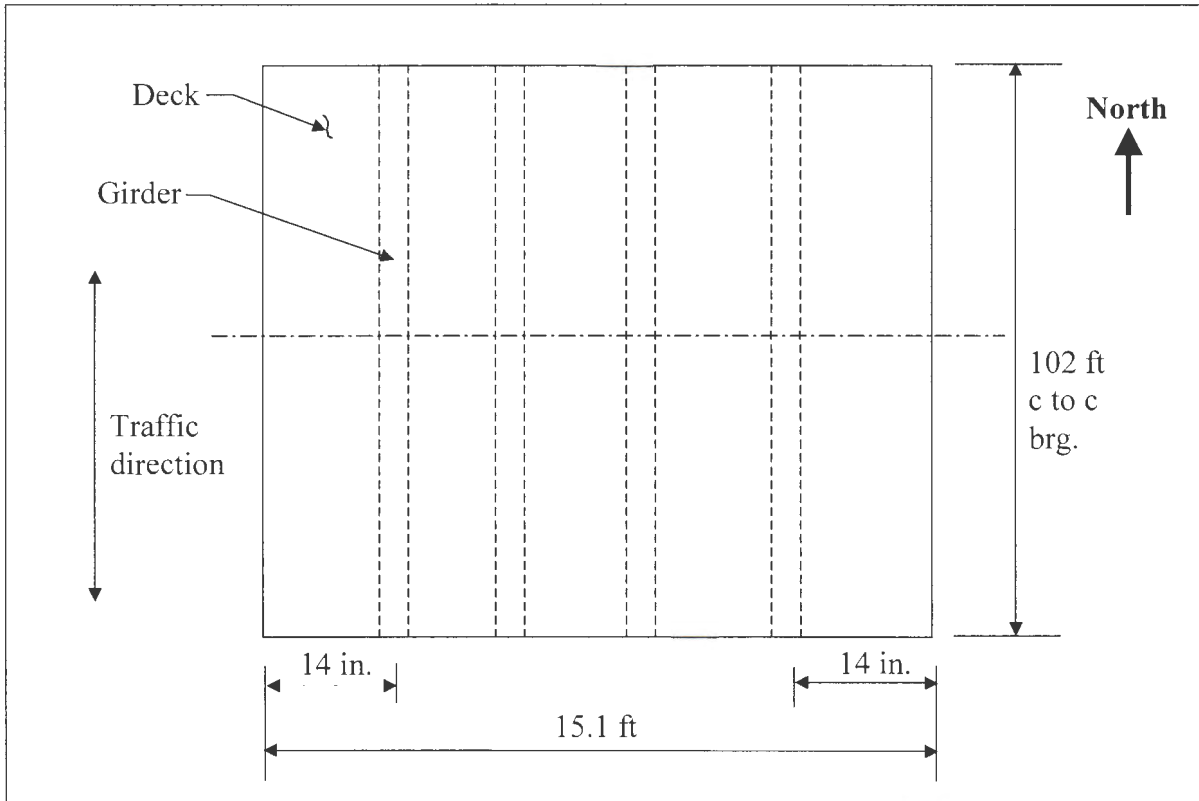
Fig. 5.19 Plan layout of the Wittson Bridge – Span 3.

5.3.2 Loading

Three load tests were done on this bridge. The load test vehicle consisted of a fully loaded three-axle dump truck with a gross vehicle weight of 55,400 pounds (Truck 1). The rear axles on Truck 1 weighed about 38,540 pounds. The rear axles weighed about 70% of the gross vehicle weight. The vehicles were positioned longitudinally on the bridge so that the centerline of the three axles was at midspan of the bridge. The transverse vehicle track width, measured center-to-center of the rear tires, was 6 feet. For Load Case 1, the longitudinal centerline of the truck coincided with the longitudinal centerline of the bridge. For Load Case 2, the longitudinal centerline of the truck was 3 feet east of the longitudinal centerline of the bridge. For Load Case 3, the longitudinal centerline of the truck was 3 feet

west of the longitudinal centerline of the bridge. Fig. 5.20 shows the configuration of the truck and the load positions on the bridge.

5.3.3 Parametric study

A parametric study on the Wittson Bridge was done for Load Case 1. The input data described in Section 5.3.1 was used to make an initial run on the program. The longitudinal modulus of elasticity of the girders was then varied and a comparative plot for girder deflections at midspan of bridge was made. Comparison was also done with respect to the experimental data from field tests. Similar deflection comparison was made by changing the end restraint. Figs. 5.24 and 5.25 show the comparative plots for the full parametric study.
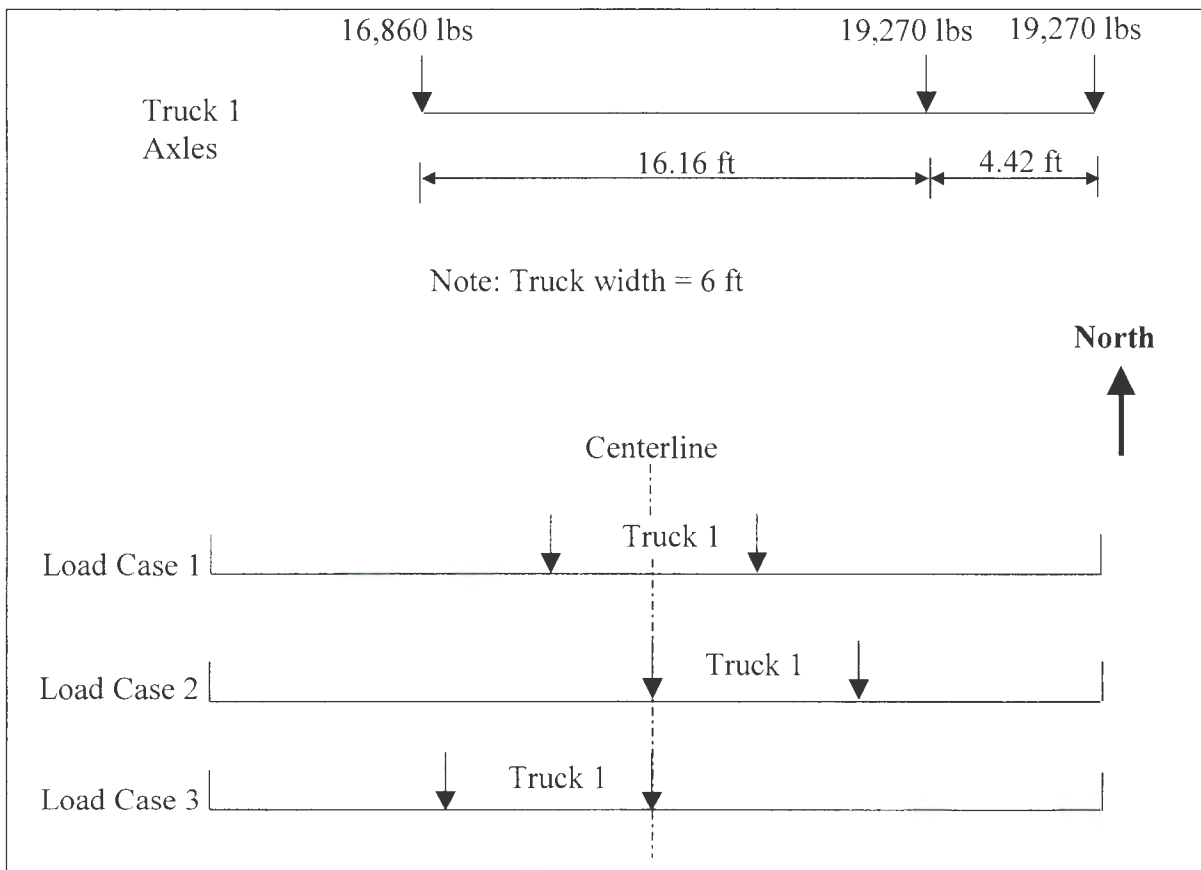
Fig. 5.20 Configuration of trucks and load positions for various load cases.

A comparative study of experimental values and analytical results for midspan girder deflections was also done for all the other load cases. The deflection plots for this comparison are shown in Figs. 5.26 and 5.27. A mesh sensitivity study was also done on the analytical model of the bridge to determine the adequate size of the element to be used. The analytical model of the Wittson County Bridge – Span 3 is shown in Fig. 5.50.

5.3.4 Comparison of maximum design stress and maximum stress from analysis

The maximum design stress for the girders on the Wittson Bridge – Span 3 was computed using the procedure described in Section 4.2.2. The maximum girder bending stress at midspan obtained using the program was 600 psi. The maximum design stress was equal to 697 psi and was calculated with the following input:

DF = 0.7 wheel-lines per girder   Width of girder = 10.625 in
M_{WL} = 7030.4 in-kips   Depth of girder = 63.125 in
M_{LL} = 4921 in-kips S_x = 7056 in^3
M_{DL} = 0 in-kips f_b = 0.697 ksi = 697 psi

From the stress comparison, it can be concluded that the finite element result compares well with the design stress computed from the design manual [1].

5.3.5 Results and discussion

Figs. 5.21 and 5.22 illustrate the results from the parametric study performed on the Wittson Bridge – Span 3, for Load Case 1. Upon varying the longitudinal modulus of elasticity of the girders from 1730 ksi to 2130 ksi, maximum deflections in the girders reduced by about 17% but there was no significant increase in the maximum stresses in the girders (Fig. 5.21). The longitudinal modulus of elasticity is a significant parameter influencing the deflection curve of the bridge.
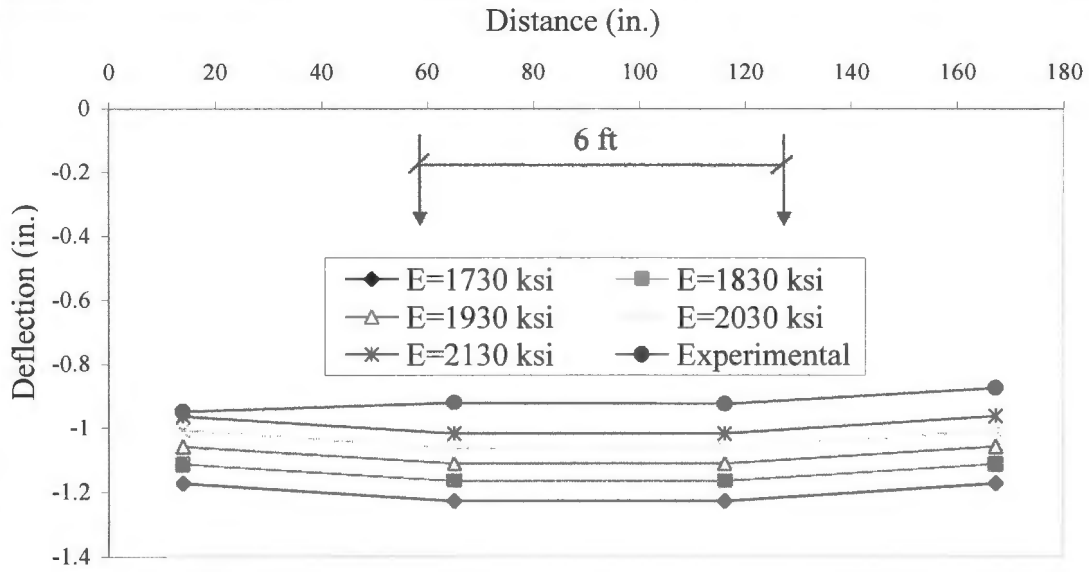
(a) Girder deflections at midspan.
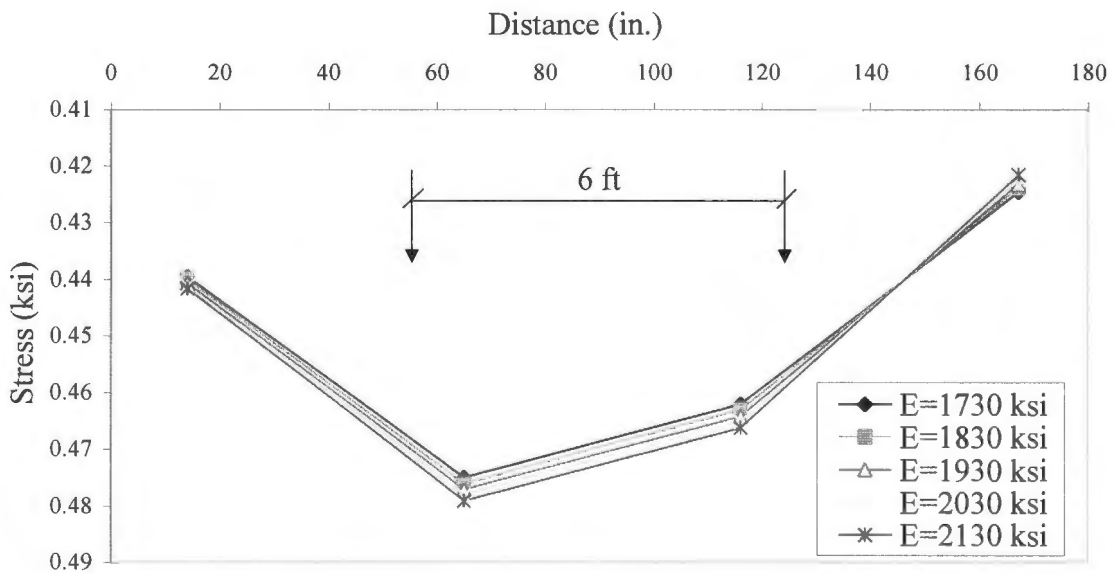
(b) Girder bending stresses at midspan.

Fig. 5.21 Comparison of girder deflections and bending stresses at midspan upon changing longitudinal modulus of elasticity of the girders for Load Case 1.

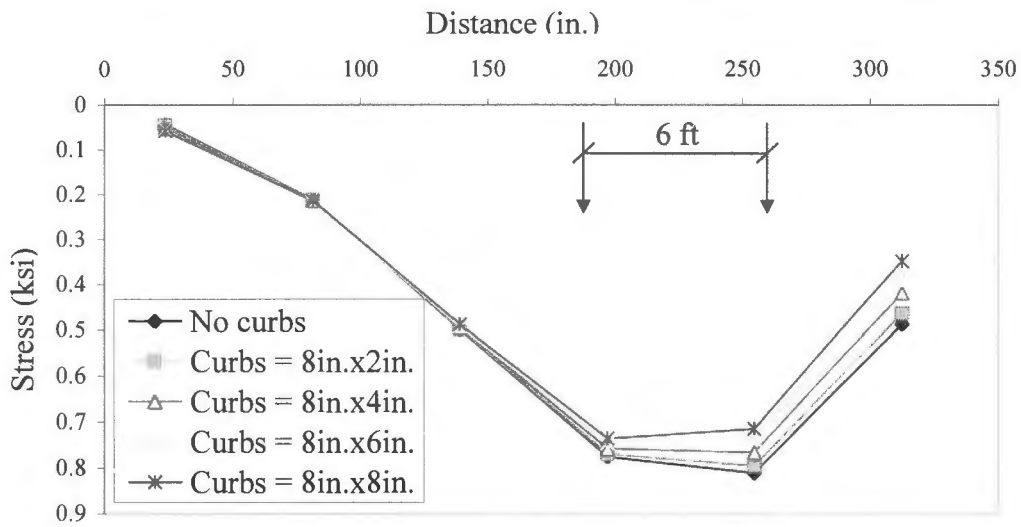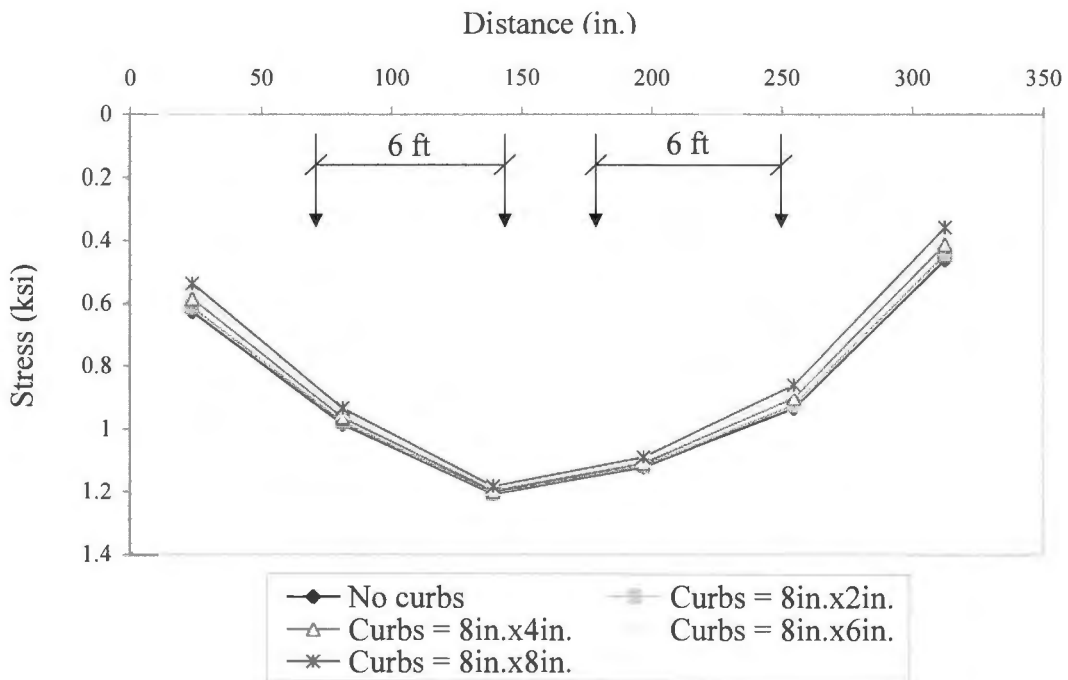**Fig. 5.22 Comparison of girder deflections at midspan upon changing support conditions for Load Case 1.**

Fig. 5.23 Comparison of girder deflections at midspan for Load Case 2.
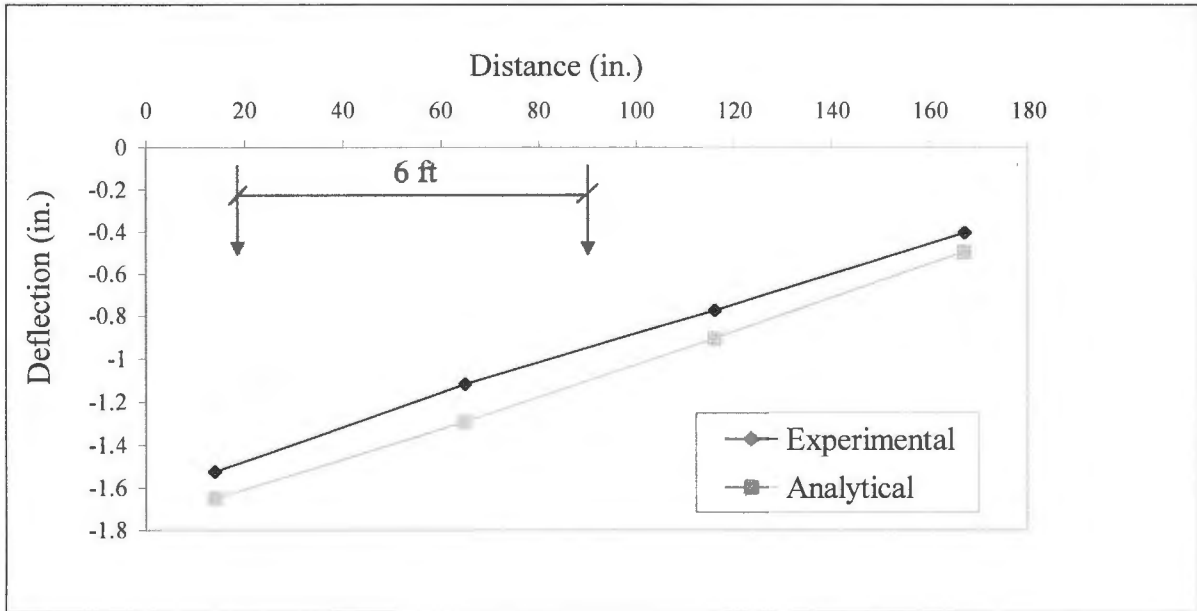
Fig. 5.24 Comparison of girder deflections at midspan for Load Case 3.

When the supports of the bridge were changed from simple supports to fixed

supports, the difference between the deflection curves was significant (Fig. 5.22). Since the

experimental deflections, in general, were lesser than the analytical results, this may be due

to some effect from the rotational restraints, if any, at the abutments or/and due to the

assumed material properties in the model. Deflection curves for all the other load cases show

that the analytical results are comparable with the experimental observations (Figs. 5.23 and

5.24). A mesh sensitivity analysis for the Wittson Bridge – Span 3 (using Load Case 1 as the

load for the sensitivity study) indicated that when the element size was 12.25 inches by 12.25

inches, the deflection and stress results reached convergence (Fig. 5.25). The aspect ratio was

fixed close to unity for the mesh sensitivity study.
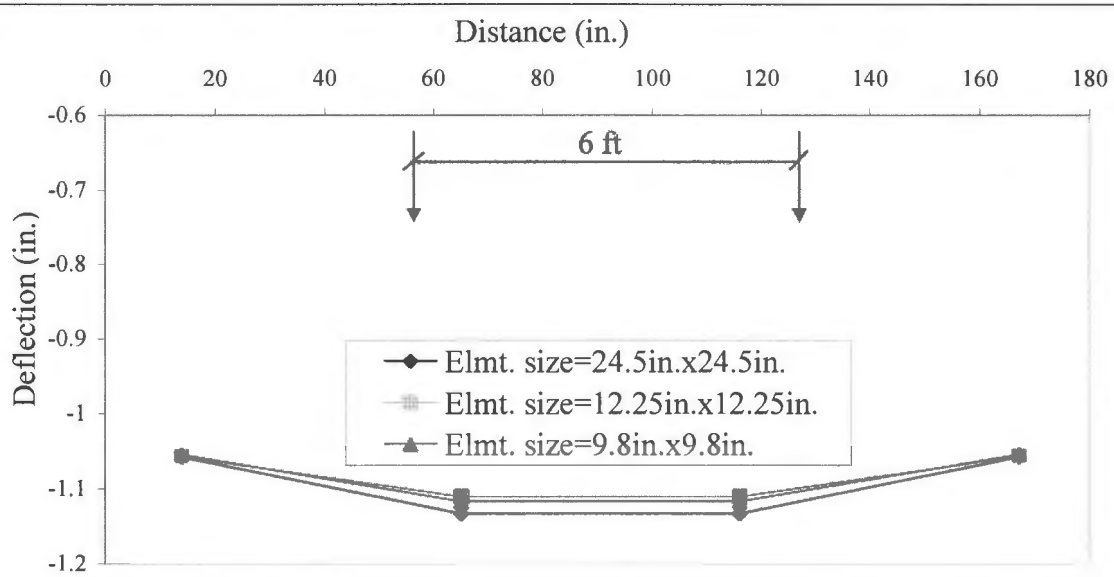
(a) Girder deflections at midspan.

(b) Girder bending stresses at midspan.

Fig. 5.25 Comparison of girder deflections and bending stresses at midspan for mesh sensitivity analysis of the Tuscaloosa Bridge – Span 3.

5.4 The Chambers County Bridge, Auburn

5.4 Description

The Chambers County Bridge is a 51.5 feet (center-to-center of bearings) long, single span, two-lane bridge. A photo of the bridge appears in Fig. 5.26. The bridge is made of Southern Yellow Pine with six girders spaced at 60 inches on center, supporting the deck. The deck panels are about 4 feet wide, 27 feet long, and 5.125 inches thick. These panels are connected to the girders by lag screws at 6 inches from each edge of the panel.

The girders have a nominal width of 8.75 inches and a nominal thickness of 43 inches. Steel guardrail on timber posts are installed on both sides of the bridge. The material properties for the analytical model were obtained from the AITC-117 Design Manual [10] for the particular grade of lamination used on bridge. The deck panels have a longitudinal and transverse modulus of elasticity of 1930 ksi and 240 ksi, respectively. The shear modulus of the deck panels is about 106 ksi. The girders have a longitudinal and transverse modulus of elasticity of 1930 ksi and 240 ksi, respectively. The shear modulus of the girders is about 106 ksi. Refer to Fig. 5.27 for the design configuration of the bridge.

5.4.2 Loading

Six load tests were done on this bridge. The load test vehicles consisted of two fully loaded three-axle dump trucks with gross vehicle weights of 59,340 pounds (Truck 1) and 63,880 pounds (Truck 2). The rear axles on Truck 1 weighed about 45,240 pounds while the rear axles on Truck 2 weighed about 49,600 pounds. The rear axles weighed about 75% of the respective gross vehicle weights. The vehicles were positioned longitudinally on the bridge so that the centerline of the three axles was centered at midspan of the bridge. The

(a) Elevation view of the Chambers County Bridge.

(b) End view of the Chambers County Bridge showing the girders.

Fig. 5.26 The Chambers County Bridge in Auburn.

transverse vehicle track width, measured center-to-center of the rear tires, was 6 feet. For Load Case 1, Truck 1 was placed on the east lane of the bridge, 2 feet away from the longitudinal centerline of the bridge. For Load Case 3, Truck 2 was placed on the west lane of the bridge, 2 feet away from the longitudinal centerline of the bridge. For Load Case 4, Truck 1 was placed 27 inches from the east edge of the bridge while for Load Case 6, Truck 2 was placed 27 inches from the west edge of the bridge. Load Case 2 was a combination of Load Cases 1 and 3, while Load Case 5 was a combination of Load Cases 4 and 6. Fig. 5.31 shows the configuration of the trucks and the load positions on the bridge.
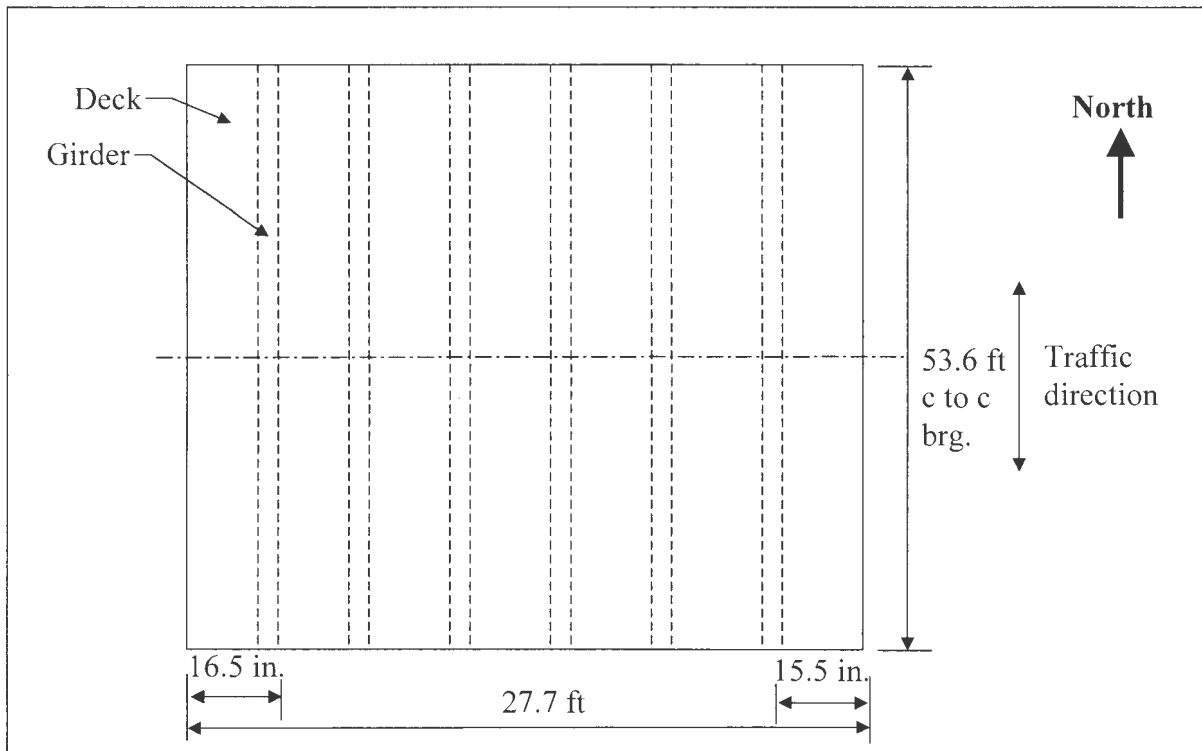
Fig. 5.27 Plan layout of the Chambers County Bridge.

Fig. 5.28 Configuration of trucks and load positions for various load cases.

5.4.3 Parametric study

A parametric study on the Chambers County Bridge was done for Load Cases 1 and 2. The input data described in Section 5.4.1 was used to make an initial run on the program. The longitudinal modulus of elasticity of the girders was then varied and a comparative plot for girder deflections at midspan of bridge was made. Comparison was also done with respect to the experimental data from field tests. Similar deflection comparison was made by changing the end restraint. Figs. 5.29 to 5.31 show the comparative plots for the full parametric study.

A comparative study of experimental values and analytical results for midspan girder deflections was also done for all the other load cases. The deflection plots for this comparison are shown in Figs. 5.32 to 5.35. A mesh sensitivity study was also done on the analytical model of the bridge to determine the adequate size of the element to be used. The analytical model of the Chambers County Bridge is shown in Fig. 5.51.

5.4.4 Comparison of maximum design stress and maximum stress from analysis

The maximum design stress for the girders on the Chambers County Bridge was computed using the procedure described in Section 4.2.2. The maximum girder bending stress at midspan obtained using the program was 930 psi. The maximum design stress was equal to 1,130 psi and was calculated with the following input:

DF = 0.9 wheel-lines per girder   Width of girder = 8.75 in
M_{WL} = 3411 in-kips   Depth of girder = 43 in
M_{LL} = 3070 in-kips S_x = 2696 in^3
M_{DL} = 0 in-kips f_b = 1.13 ksi = 1,130 psi

From the stress comparison, it can be concluded that the finite element result compares well with the design stress computed from the design manual [1].

5.4.5 Results and discussion

Figs. 5.29 to 5.31 illustrate the results from the parametric study performed on the Chambers County Bridge for Load Cases 1 and 2. Upon varying the longitudinal modulus of elasticity of the girders from 1730 ksi to 2130 ksi, maximum deflections in the girders reduced by about 15%, but there was no significant increase in the maximum stresses in the girders for both load cases (Figs. 5.29 and 5.30). The longitudinal modulus of elasticity is a significant parameter influencing the deflection curve of the bridge.

When the supports of the bridge were changed from simple supports to fixed supports, the difference between the deflection curves was significant (Fig. 5.31). Since the experimental deflections, in general, were lesser than the analytical results, this may be due to some effect from the rotational restraints, if any, at the abutments or/and due to the assumed material properties in the model. Deflection curves for all the other load cases show that the analytical results are comparable with the experimental observations (Figs. 5.32 to 5.35). A mesh sensitivity analysis for the Chambers County Bridge (using Load Case 1 as the load for the sensitivity study) indicated that when the element size was 12 inches by 12 inches, the deflection and stress results reached convergence (Fig. 5.36). The aspect ratio was fixed at unity for the mesh sensitivity study.

5.5 The Hibbard Creek Bridge, Montana

5.5.1 Skew bridges

Skew bridges are effective solutions in bridge design when problems like complex alignment occur. The analysis of skew bridges is, however, complicated by the absence of orthogonal relationships. Rigorous solutions are difficult and are rarely obtained. In addition,
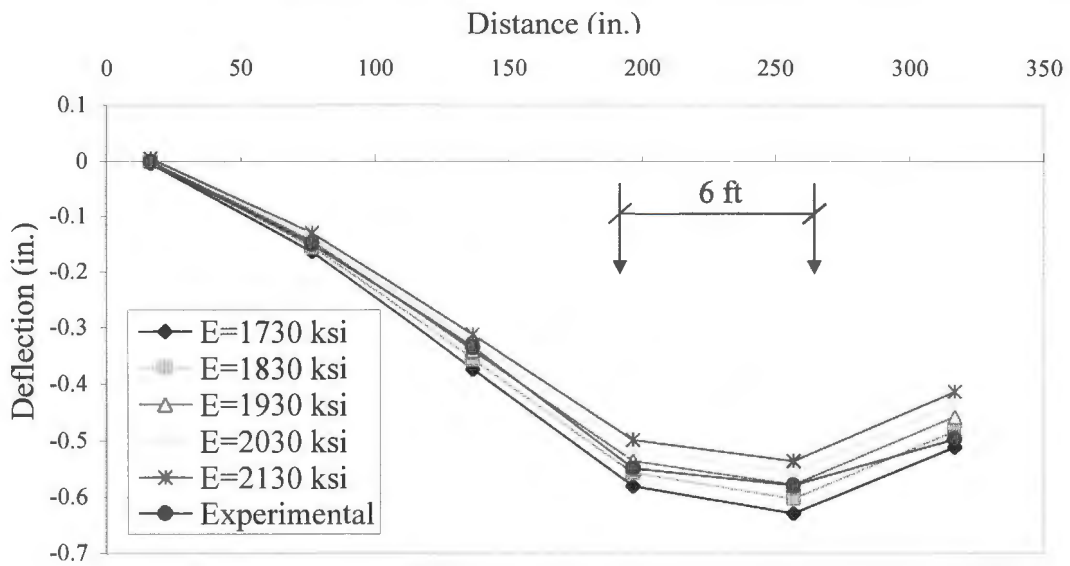
(a) Load Case 1.
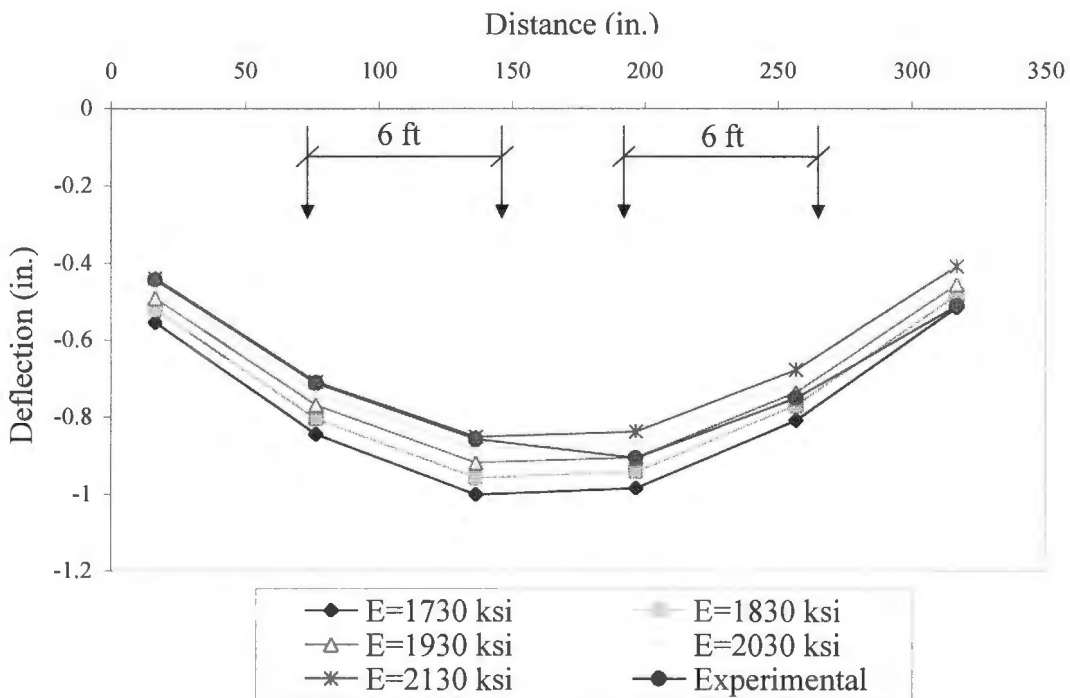
(b) Load Case 2.

Fig. 5.29 Comparison of girder deflections at midspan upon changing longitudinal modulus of elasticity of the girders for Load Cases 1 and 2.

(a) Load Case 1.
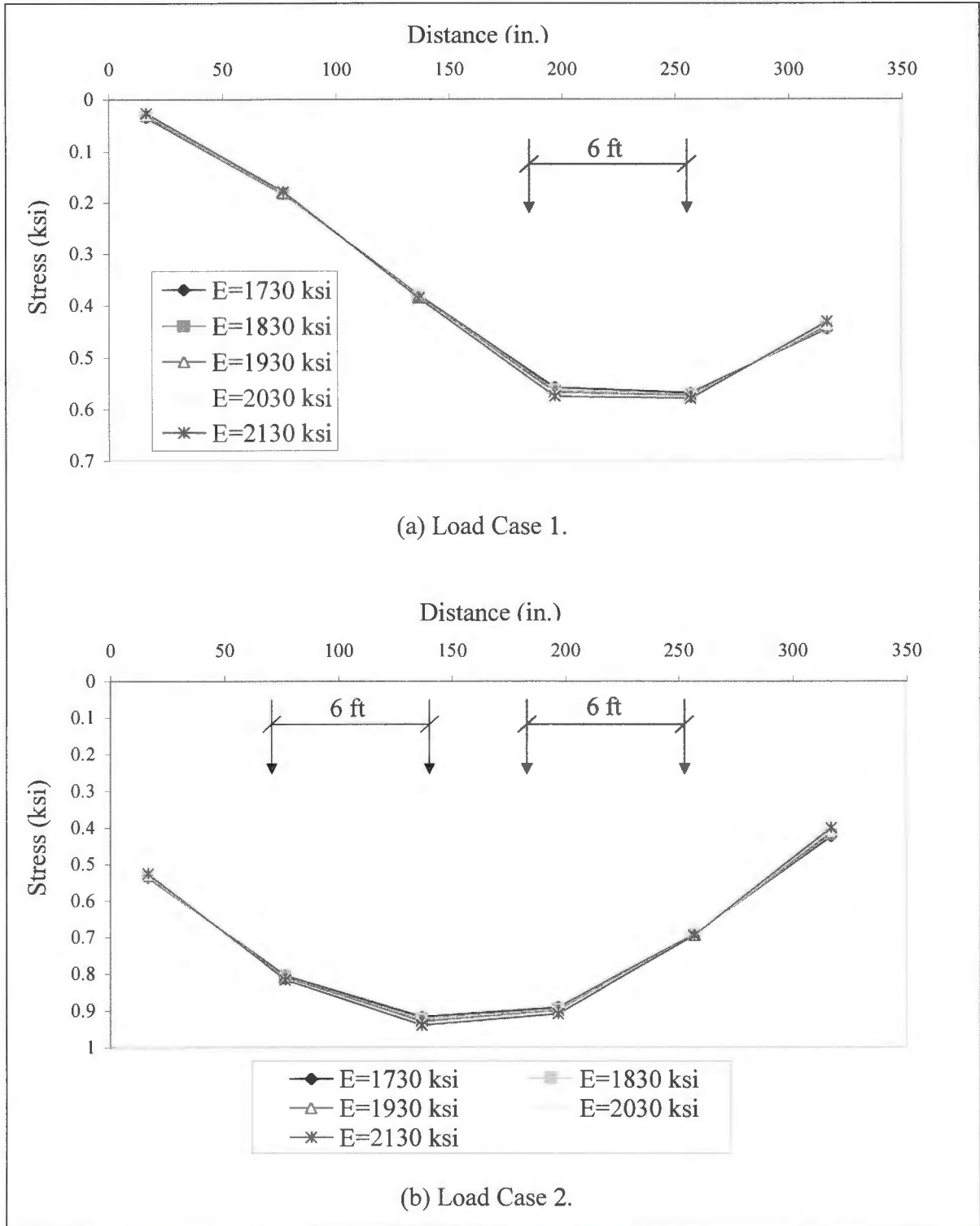
(b) Load Case 2.

Fig. 5.30 Comparison of analytical girder bending stresses at midspan upon changing longitudinal modulus of elasticity of the girders for Load Cases 1 and 2.
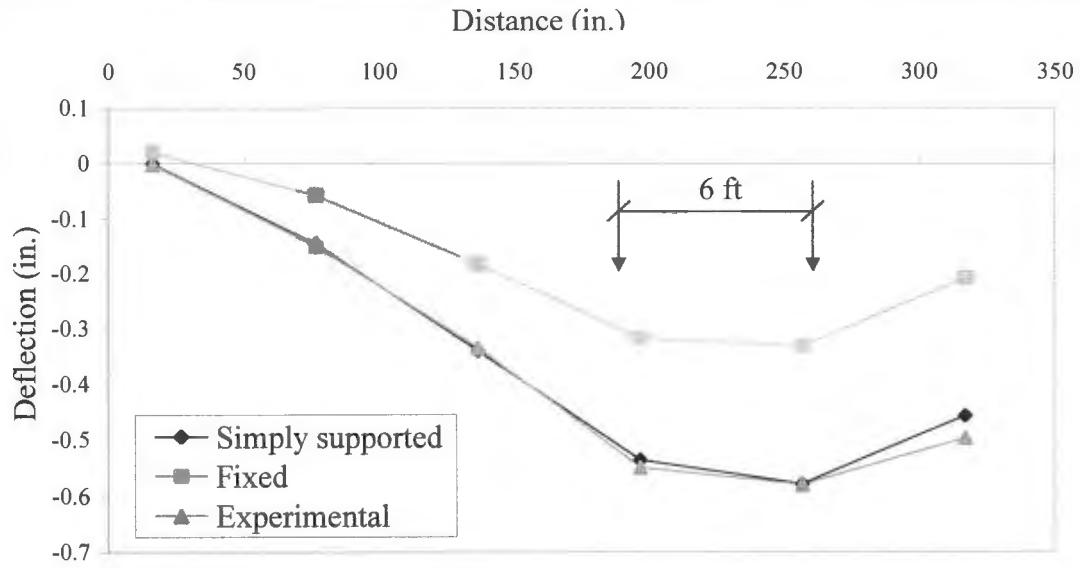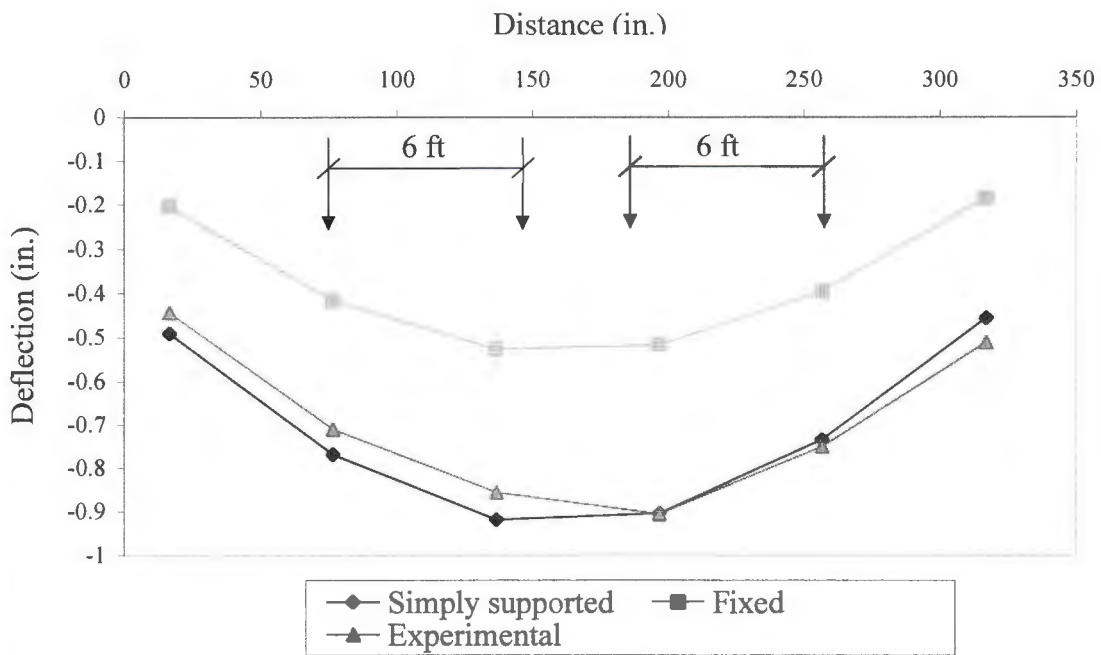
(a) Load Case 1.

(b) Load Case 2.

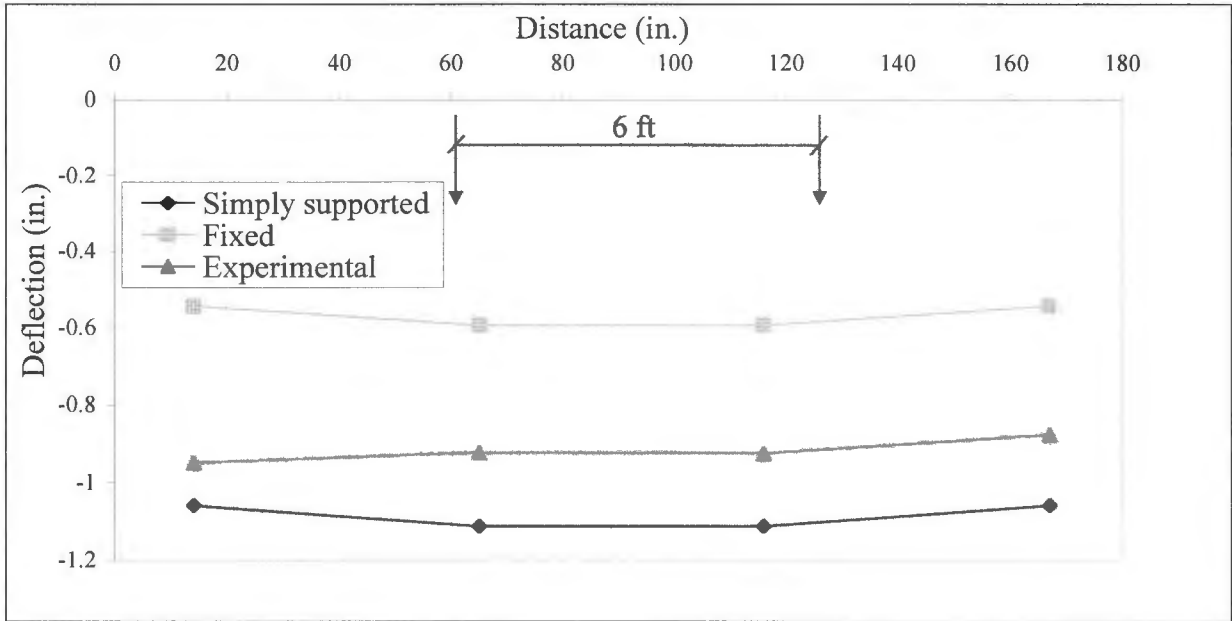**Fig. 5.31 Comparison of girder deflections at midspan upon changing support conditions for Load Cases 1 and 2.**
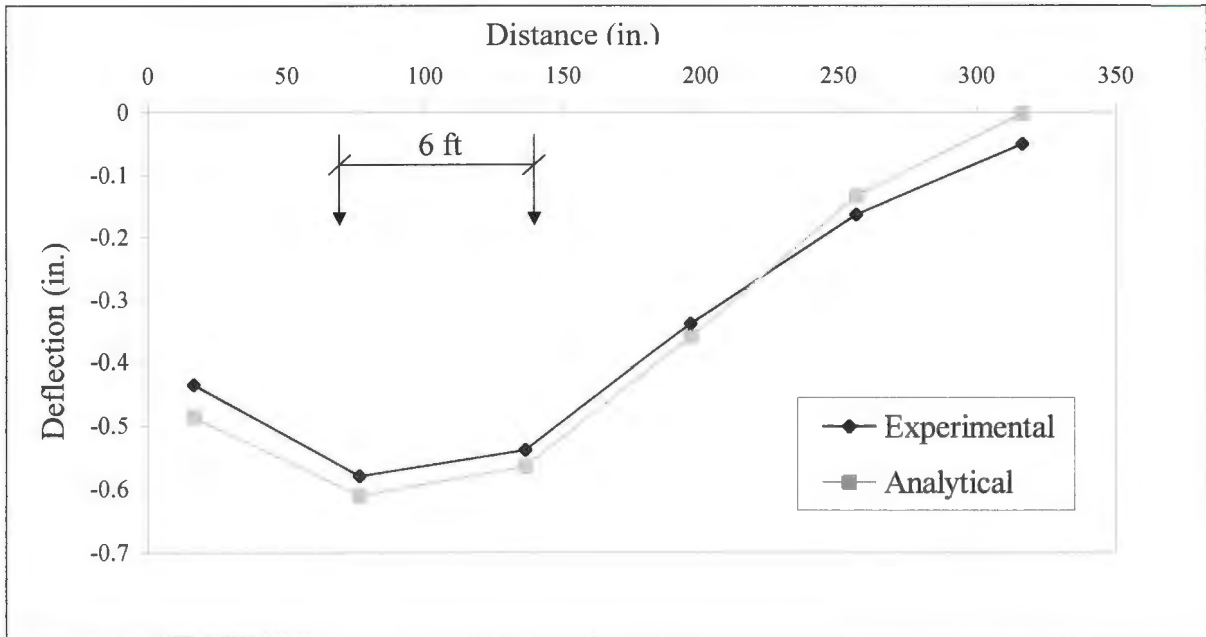
Fig. 5.32 Comparison of girder deflections at midspan for Load Case 3.
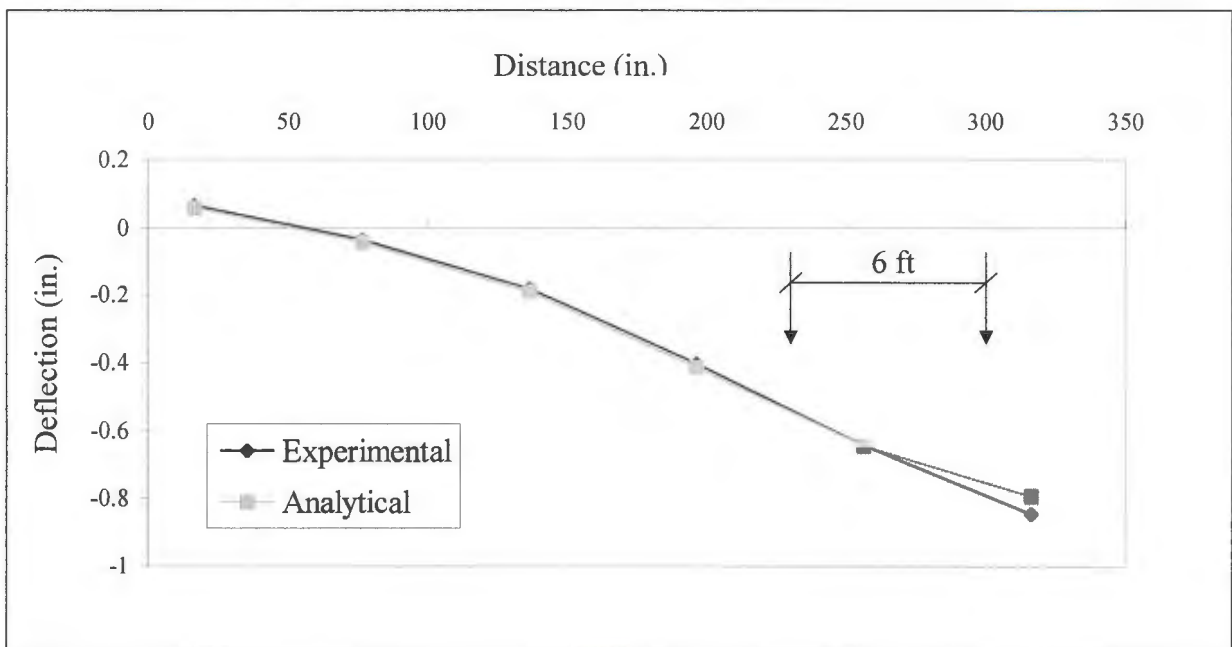
Fig. 5.33 Comparison of girder deflections at midspan for Load Case 4.
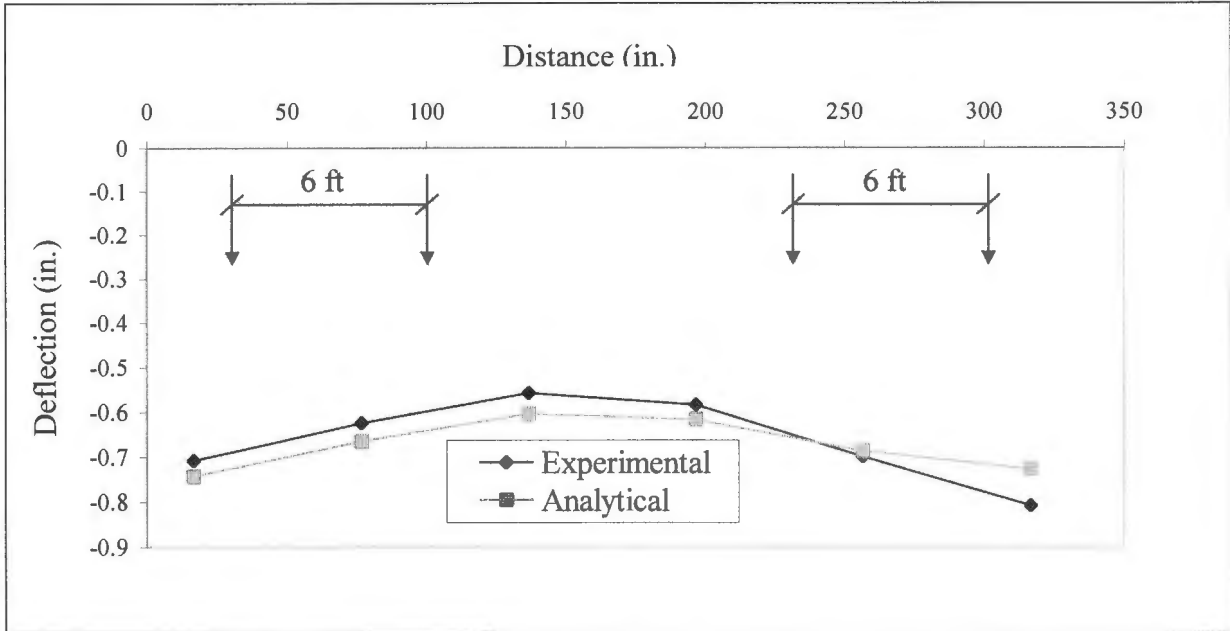
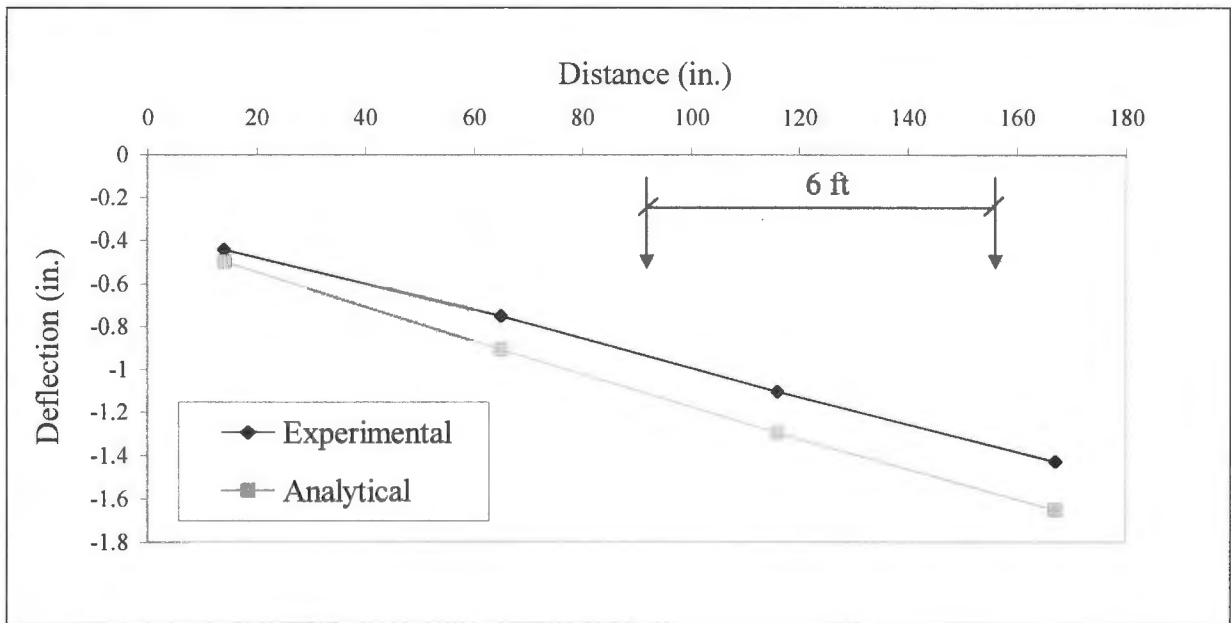**Fig. 5.34 Comparison of girder deflections at midspan for Load Case 5.**
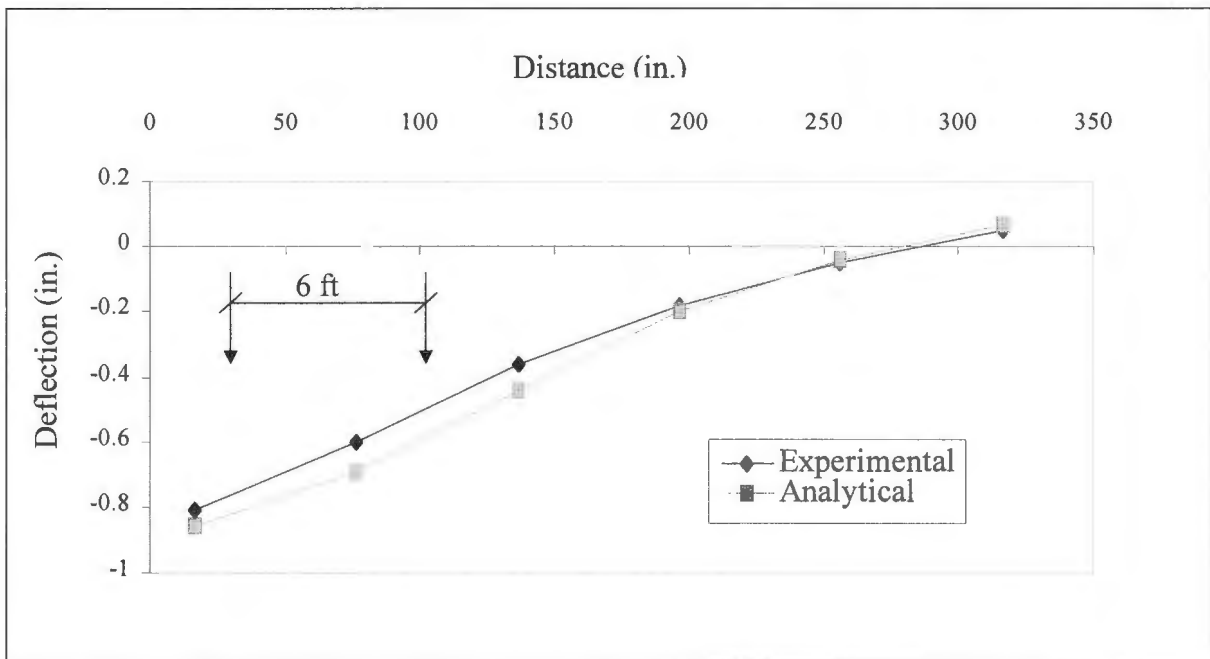
Fig. 5.35 Comparison of girder deflections at midspan for Load Case 6.
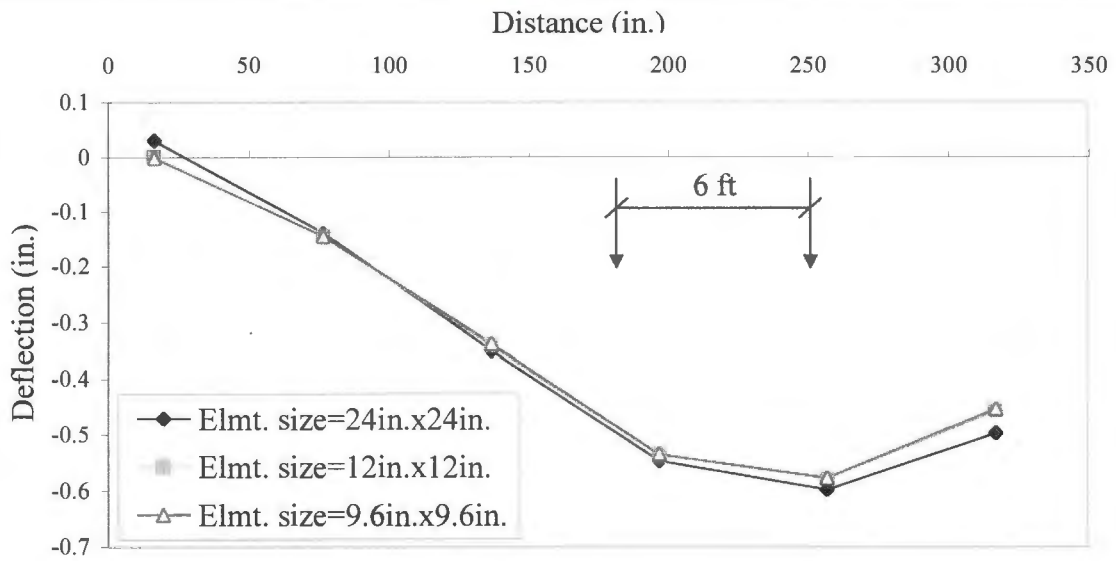
(a) Girder deflections at midspan.
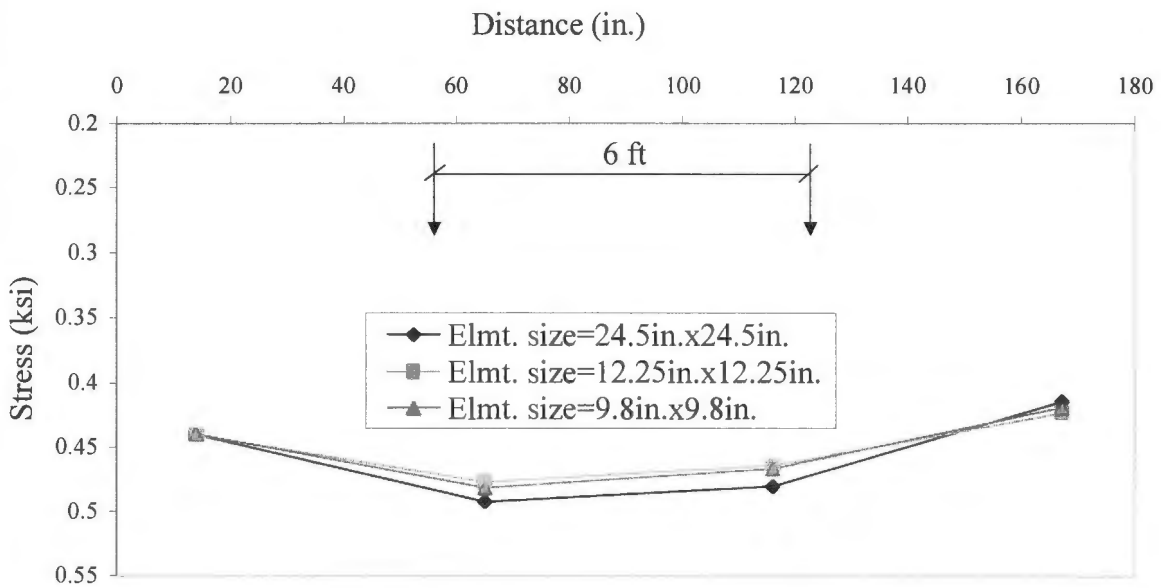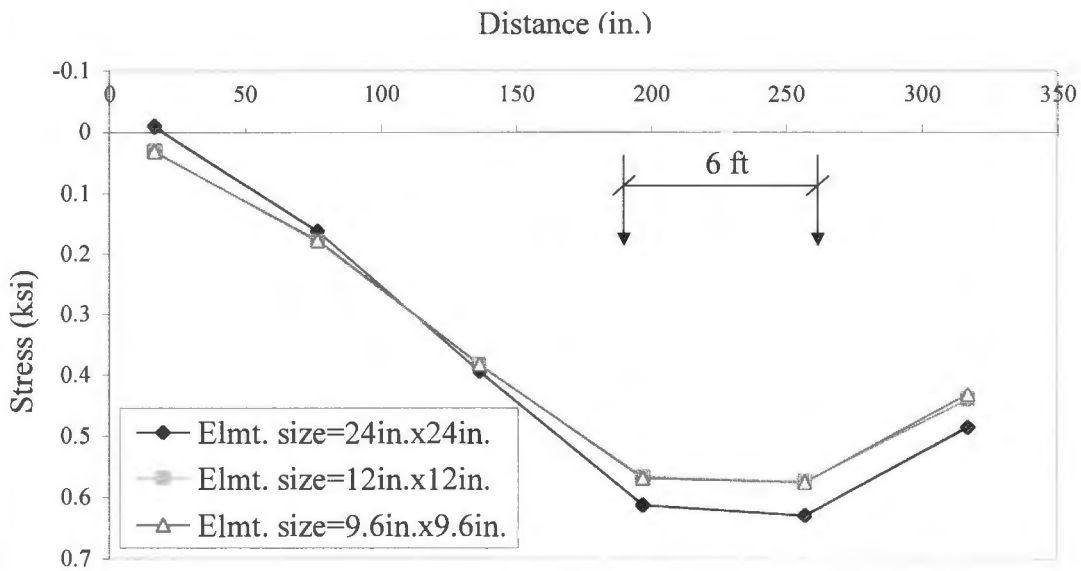
(b) Girder bending stresses at midspan.

Fig. 5.36 Comparison of girder deflections and bending stresses at midspan for mesh sensitivity analysis of the Chambers County Bridge.

singularities occurring at the obtuse corners make the exact solution even more involved. The inadequacy of analytical methods leads to extensive use of numerical techniques like the finite difference method and the finite element method. In application of both numerical approaches, however, care should be exercised in assessment of stresses at the obtuse corners, where stress concentration is known to occur [15].

5.5.2 Description

The Hibbard Creek Bridge is owned by Yellowstone County in Montana and is located on a double-lane gravel roadway accessing a 400,000-acre area encompassing 17 townships. The bridge railing system is treated timber posts and a glue laminated rail, faced with a galvanized steel w-beam. The approach guardrail system is treated timber posts with a galvanized steel w-beam. The bridge is skewed (left end ahead) by 30 degrees and is made of Coast Douglas Fir Grade 2, with six girders supporting the deck panels. The deck panels are about 4 feet wide, 32 feet long, and 5.125 inches thick and are laid transversely on the girders. They are connected to the girders by lag bolts at 6 inches in from each edge of the panel. The bridge measured 40 feet in span, measured center-to-center of bearings. The girders have a nominal width of 8.75 inches, and a nominal thickness of 28.5 inches. The curbs on the bridge have a cross-section dimension of 8 inches by 8 inches. The material properties for the analytical model were obtained from the Douglas Fir Handbook [9] for the particular grade of lamination used on bridge. The deck panels have a longitudinal and transverse modulus of elasticity of 1800 kips-per-square inch (ksi) and 130 ksi, respectively. The shear modulus of the deck panels is about 100 ksi. The girders have a longitudinal and transverse modulus of elasticity of 2000 ksi and 240 ksi, respectively. The shear modulus of

Fig. 5.37 Plan layout of the Hibbard Creek Bridge.

the girders is about 106 ksi. Refer to Fig. 5.40 for the design configuration of the bridge.

5.5.3 Loading

Six load tests were done on this bridge. The load test vehicles consisted of two fully loaded three-axle dump trucks with gross vehicle weights of 50,920 pounds (Truck 1) and 50,080 pounds (Truck 2). The rear axles on Truck 1 weighed about 36,820 pounds while the rear axles on Truck 2 weighed about 36,000 pounds. The rear axles weighed about 75% of the respective gross vehicle weights. The vehicles were positioned longitudinally on

Fig. 5.38 Configuration of trucks and load positions for various load cases.

the bridge so that the two rear axles were centered at midspan of the bridge (for the skew bridge, the midspan was considered to be the skew centerline of the bridge). The transverse vehicle track width, measured center-to-center of the rear tires, was 6 feet. The load cases are similar to the cases described in Section 5.2.2. Fig. 5.41 shows the configuration of the trucks and the load positions for various load cases.
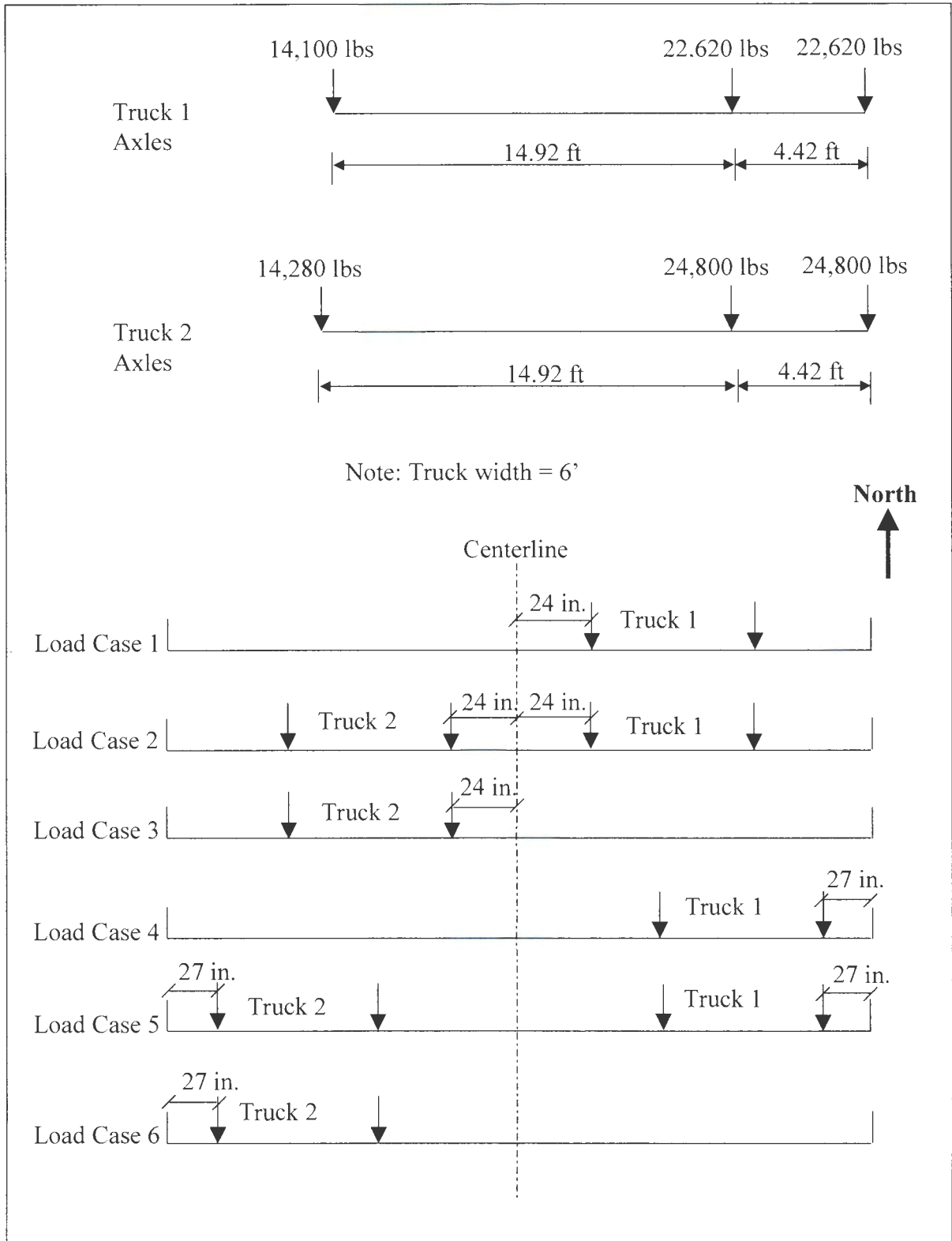
5.5.4 Parametric study

A parametric study on the Hibbard Creek Bridge was done for Load Cases 1 and 3. The input data described in Section 5.5.2 was used to make an initial run on the program. The longitudinal modulus of elasticity of the girders was then varied and a comparative plot for girder deflections at midspan of bridge was made. Comparison was also done with respect to the experimental data from field tests. Similar deflection comparisons were made by individually changing the curb height, and end restraint. Figs. 5.42 to 5.46 shows the comparative plots for the full parametric study.

A comparative study of experimental values and analytical results for midspan girder deflections was also done for all the other load cases. The deflection plots for this comparison are shown in Figs. 5.47 to 5.50. A mesh sensitivity study was also done on the analytical model of the bridge to determine the adequate size of the element to be used. The analytical model of the Hibbard Creek Bridge is shown in Fig. 5.52.

5.5.5 Comparison of maximum design stress and maximum stress from analysis

The maximum design stress for the girders on the Hibbard Creek Bridge was computed using the procedure described in Section 4.2.2. The maximum girder bending

stress at midspan obtained using the program was 1,320 psi. The maximum design stress was equal to 1,560 psi and was calculated with the following input:

DF = 0.96 wheel-lines per girder   Width of girder = 8.75 in
M_{WL} = 1926 in-kips   Depth of girder = 28.5 in
M_{LL} = 1849 in-kips S_x = 1185 in^3
M_{DL} = 0 in-kips f_b = 1.56 ksi = 1,560 psi

From the stress comparison, it can be concluded that the finite element result compares well with the design stress computed from the design manual [1].

5.5.6 Results and discussion

Figs. 5.42 to 5.46 illustrate the results from the parametric study performed on the Hibbard Creek Bridge for Load Cases 1 and 3. Upon varying the longitudinal modulus of elasticity of the girders from 1800 ksi to 2200 ksi, maximum deflections in the girders reduced by about 14% and maximum stresses in the girders increased by about 5% for both Load Cases 1 and 3 (Figs. 5.42 and 5.43). The longitudinal modulus of elasticity is a significant parameter influencing the deflection curve of the bridge. Deflection and stress curves also showed a significant change near the edges of the bridge when the curb dimensions were varied (Figs. 5.44 and 5.45). The changes as we moved towards the center of the bridge were minimal; thus, we can conclude that the effect of curbs is restricted to regions near the edge of the bridge. Upon varying curb height from 0 to 8 inches, maximum deflections and stresses in the exterior girders decreased by 30% for Load Case 1, and by 22% for Load Case 3.

When the supports of the bridge were changed from simple supports to fixed supports, the difference between the deflection curves was significant (Fig. 5.46). Since the experimental deflections, in general, were lesser than the analytical results, this may be due

to some effect from the rotational restraints, if any, at the abutments or/and due to the assumed material properties in the model. Deflection curves for all the other load cases show that the analytical results are comparable with the experimental observations (Figs. 5.47 to 5.50). A mesh sensitivity analysis for the Hibbard Creek Bridge (using Load Case 1 as the load for the sensitivity study) indicated that when the element size was 12 inches by 12 inches, the deflection and stress results reached convergence (Fig. 5.51). The aspect ratio was fixed close to unity for the mesh sensitivity study.

(a) Load Case 1.
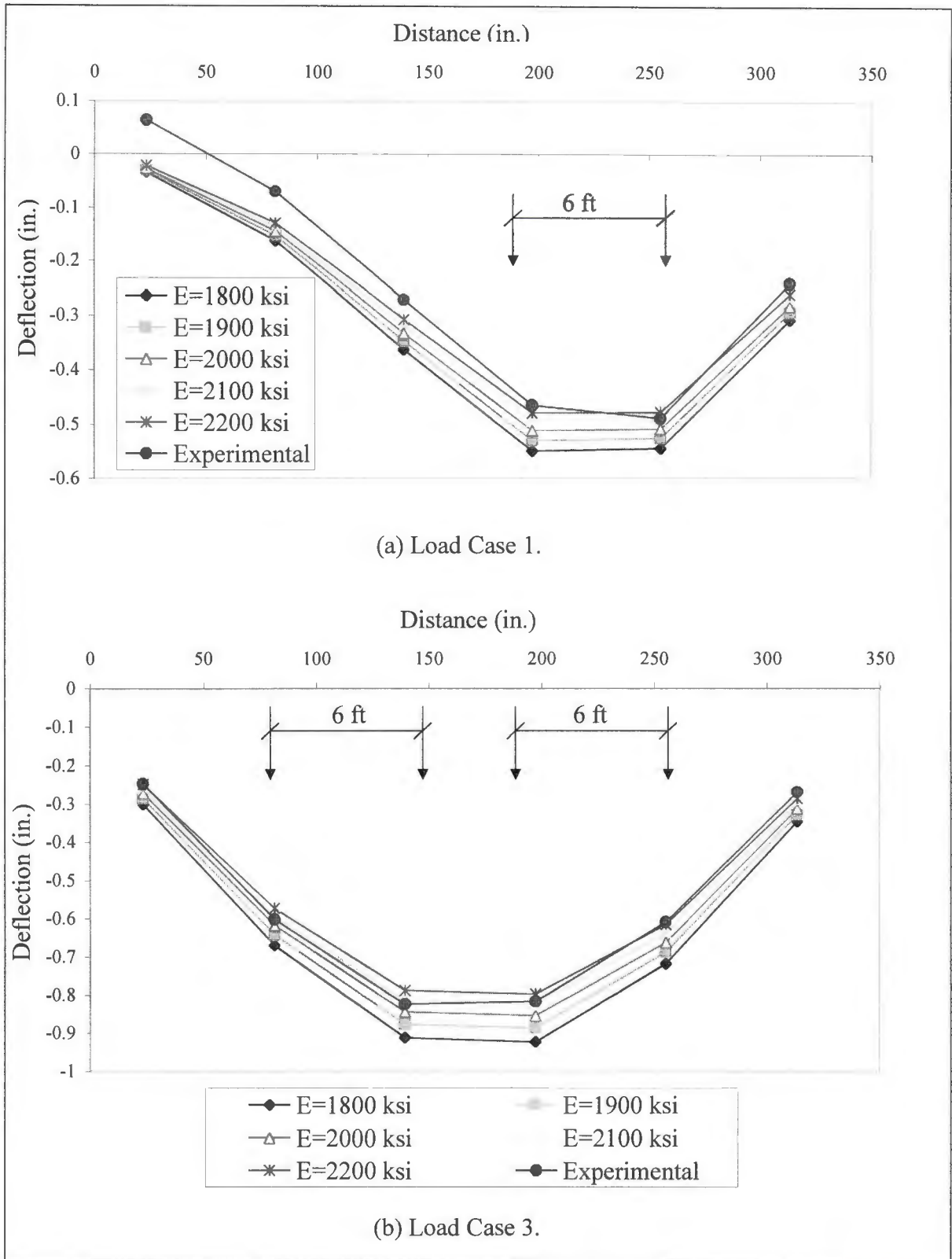
(b) Load Case 3.

Fig. 5.39 Comparison of girder deflections at midspan upon changing longitudinal modulus of elasticity of the girders for Load Cases 1 and 3.
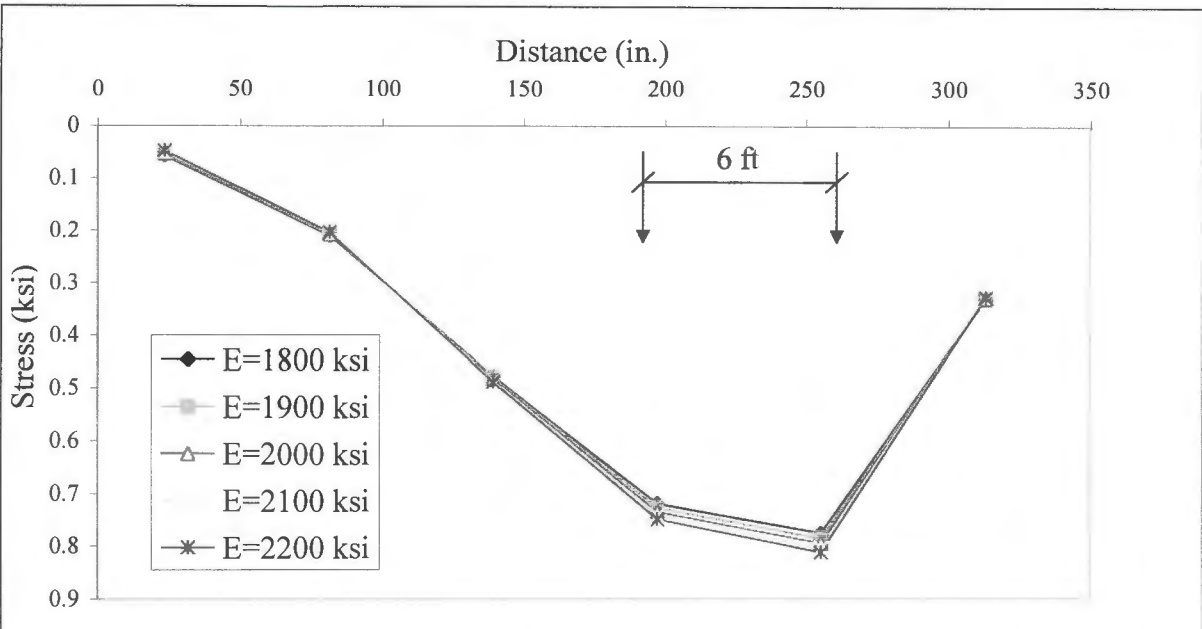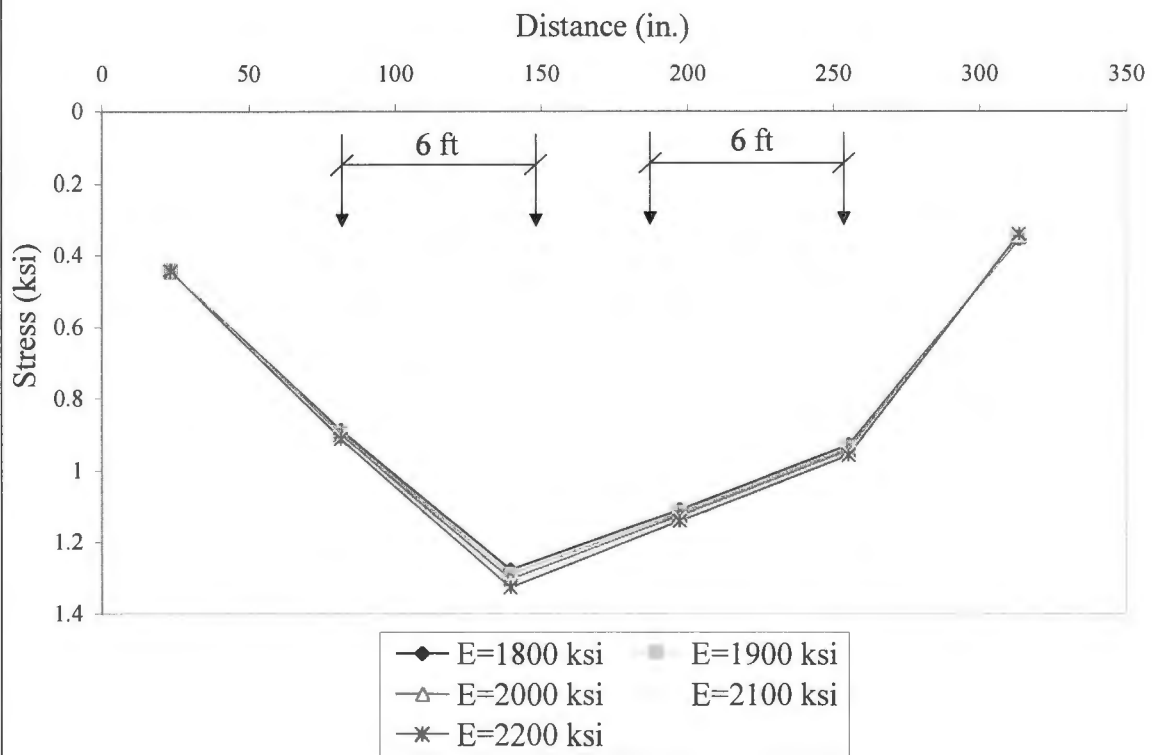
(a) Load Case 1.

(b) Load Case 3.

Fig. 5.40 Comparison of analytical girder bending stresses at midspan upon changing longitudinal modulus of elasticity of the girders for Load Cases 1 and 3.

(a) Load Case 1.
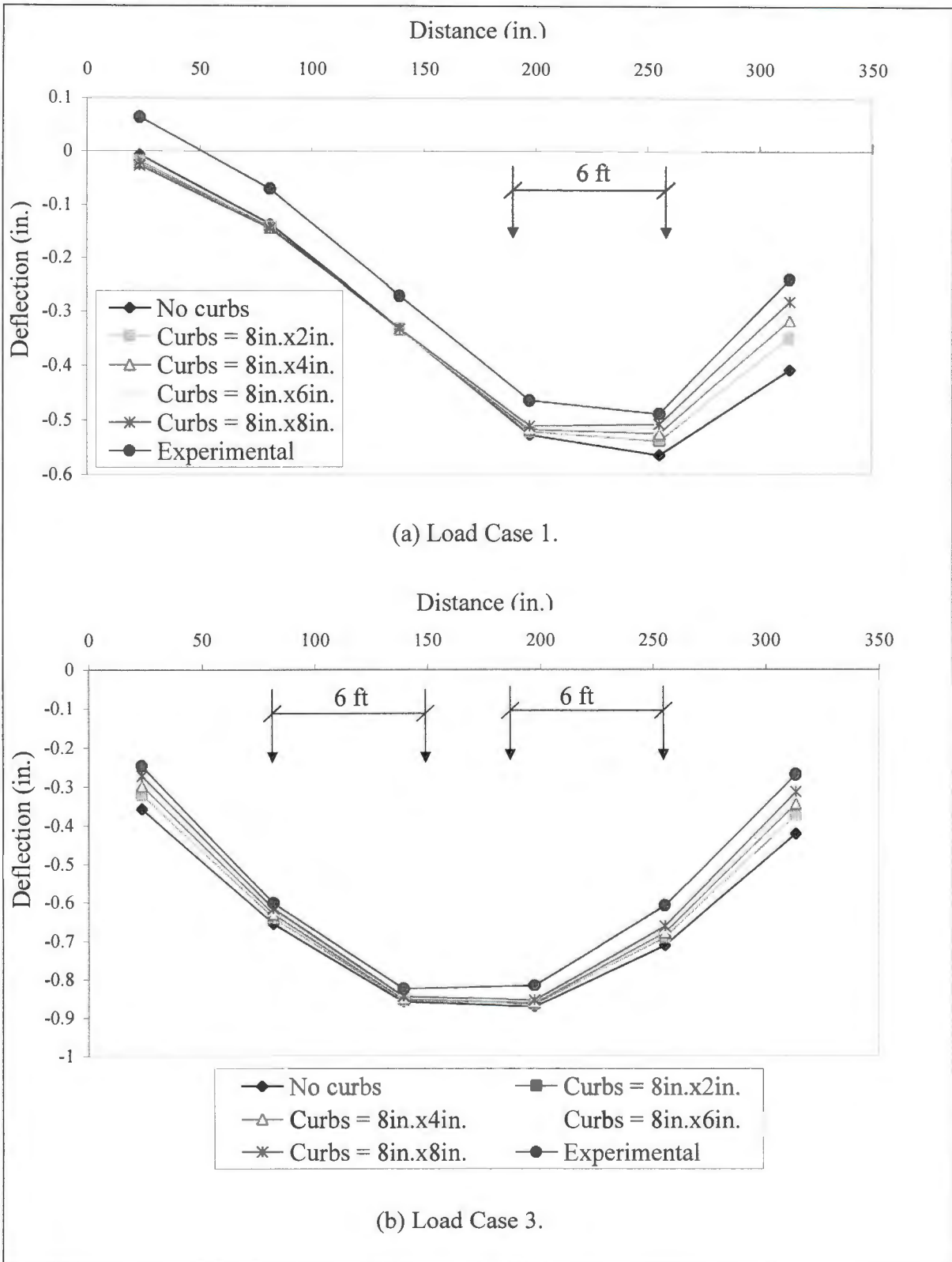
(b) Load Case 3.

Fig. 5.41 Comparison of girder deflection at midspan upon varying curb height for Load Cases 1 and 3.

(a) Load Case 1.
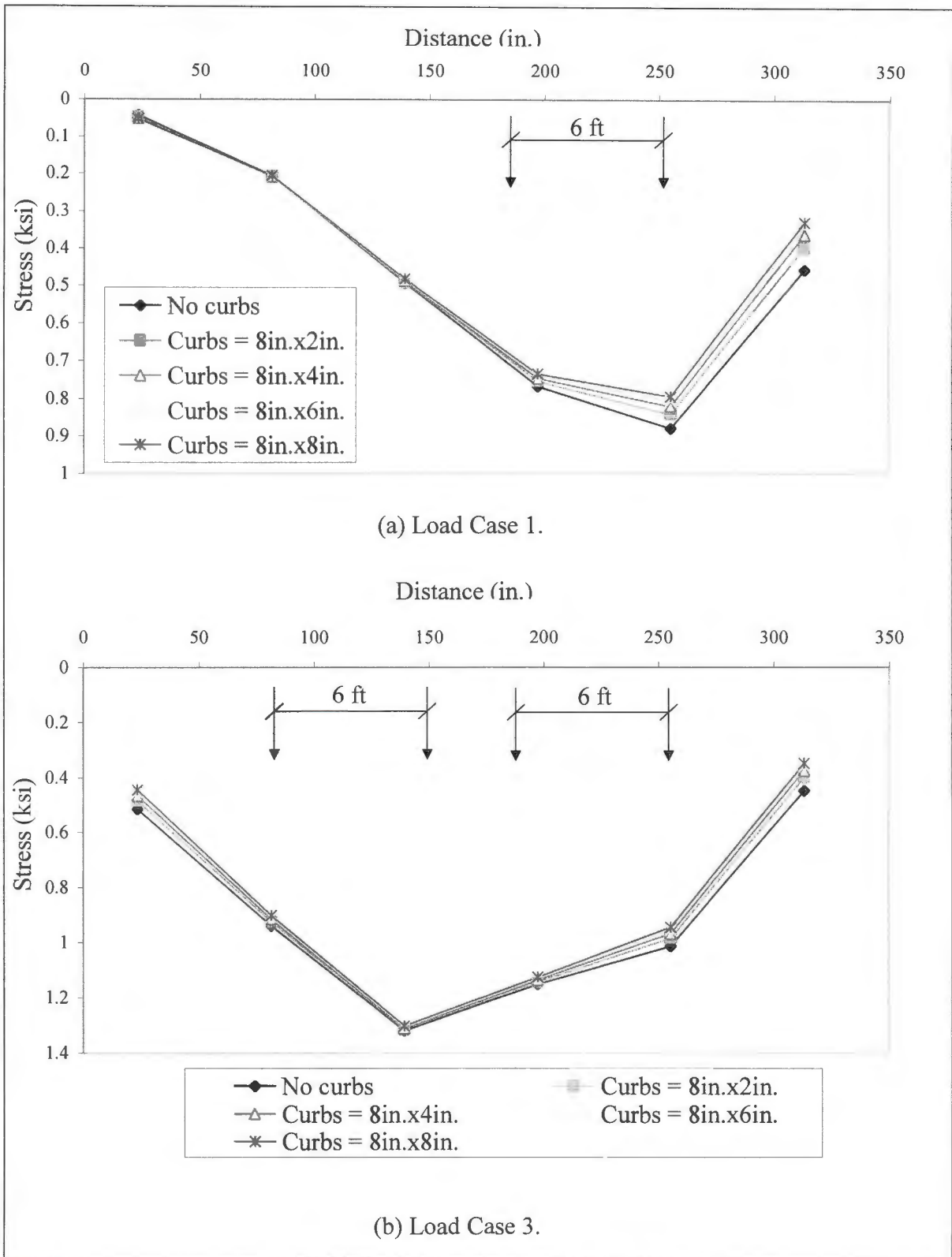
(b) Load Case 3.

Fig. 5.42 Comparison of analytical girder bending stresses at midspan upon varying curb height for Load Cases 1 and 3.
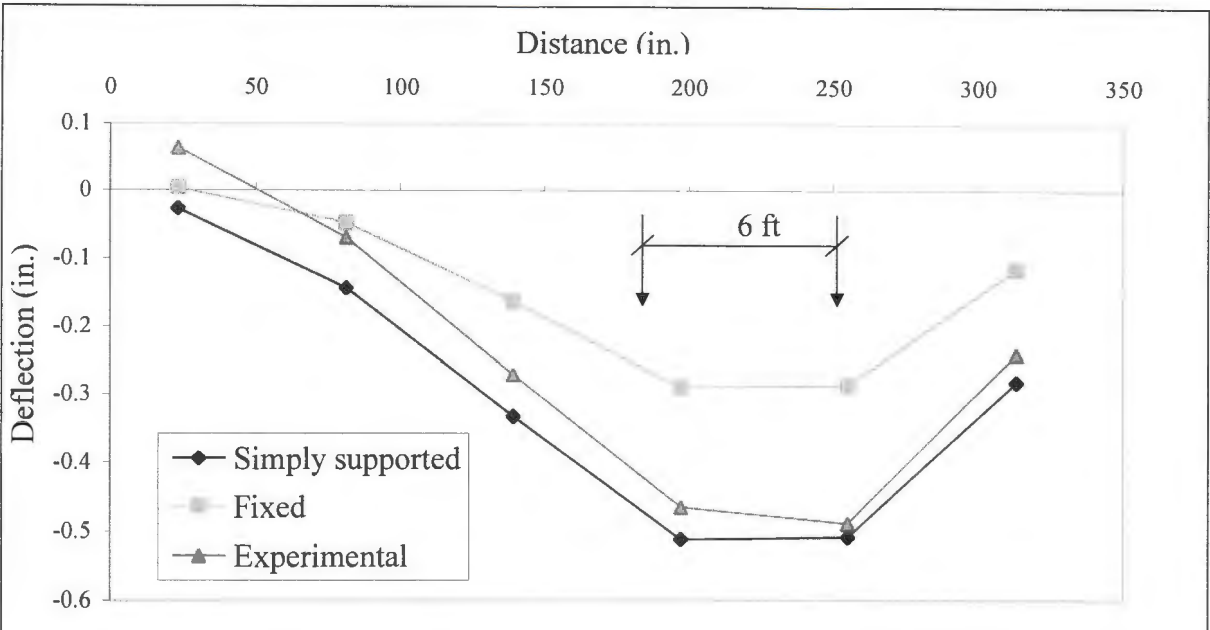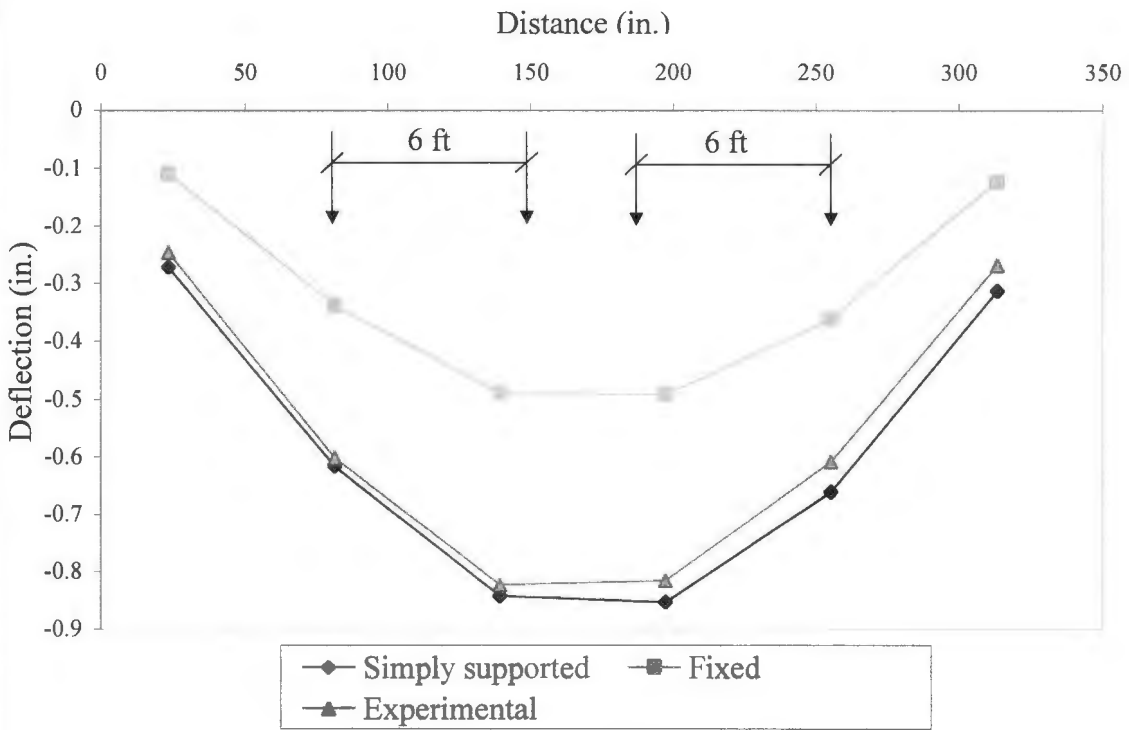
(a) Load Case 1.

(b) Load Case 3.

Fig. 5.43 Comparison of girder deflections at midspan upon changing support conditions for Load Cases 1 and 3.
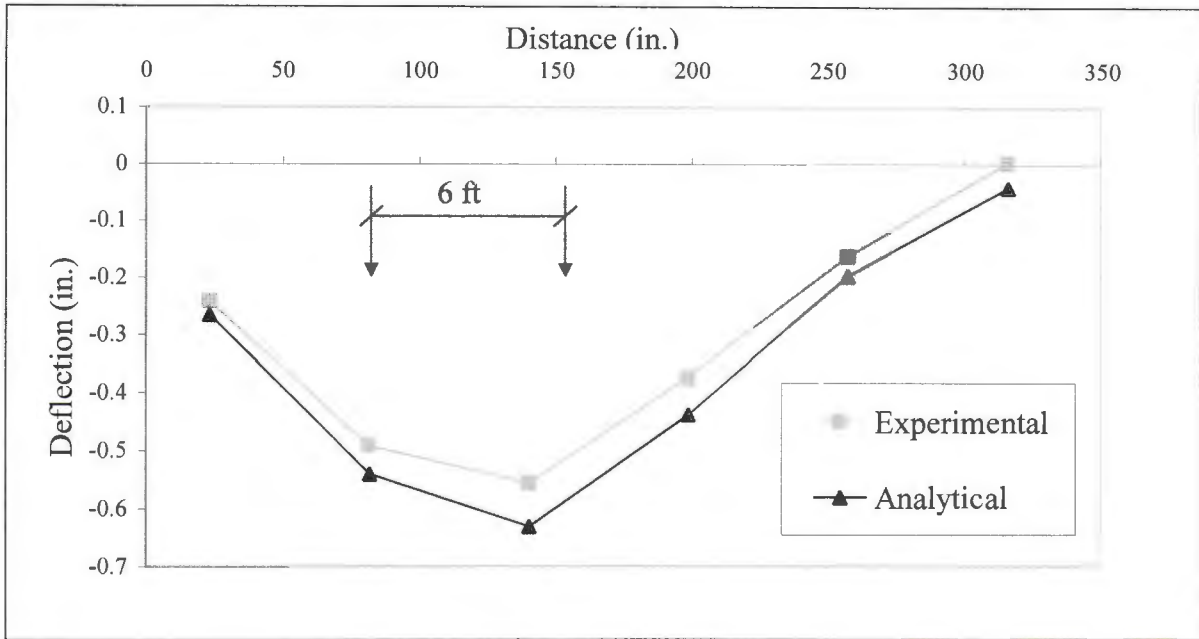
Fig. 5.44 Comparison of girder deflections at midspan for Load Case 2.
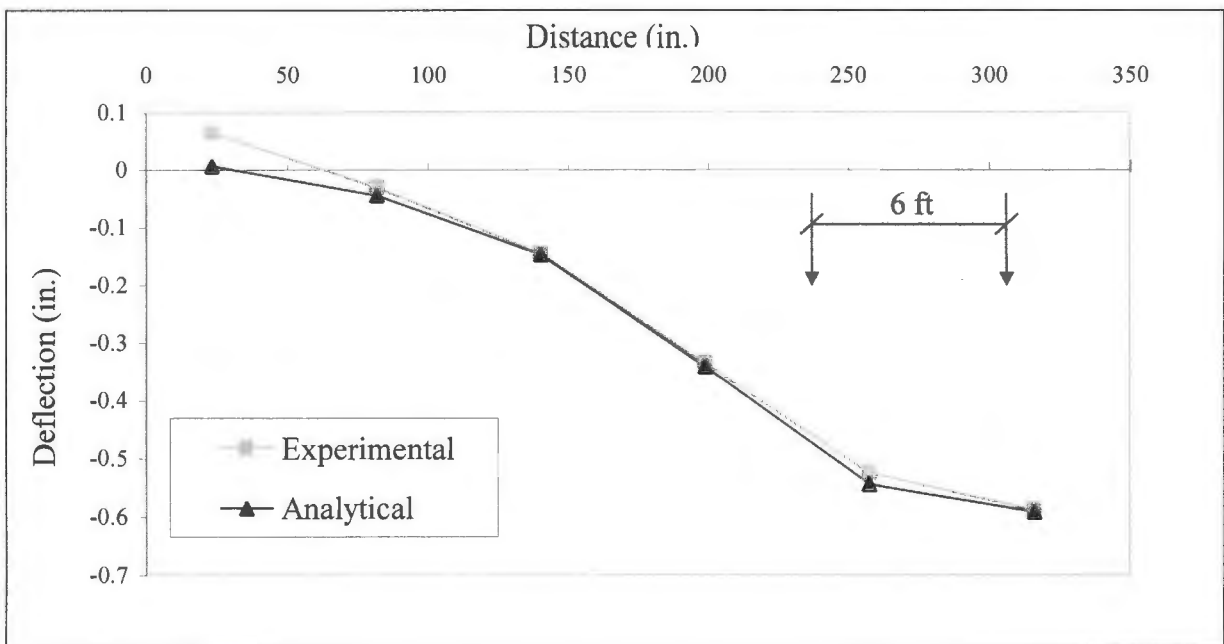
Fig. 5.45 Comparison of girder deflections at midspan for Load Case 4.
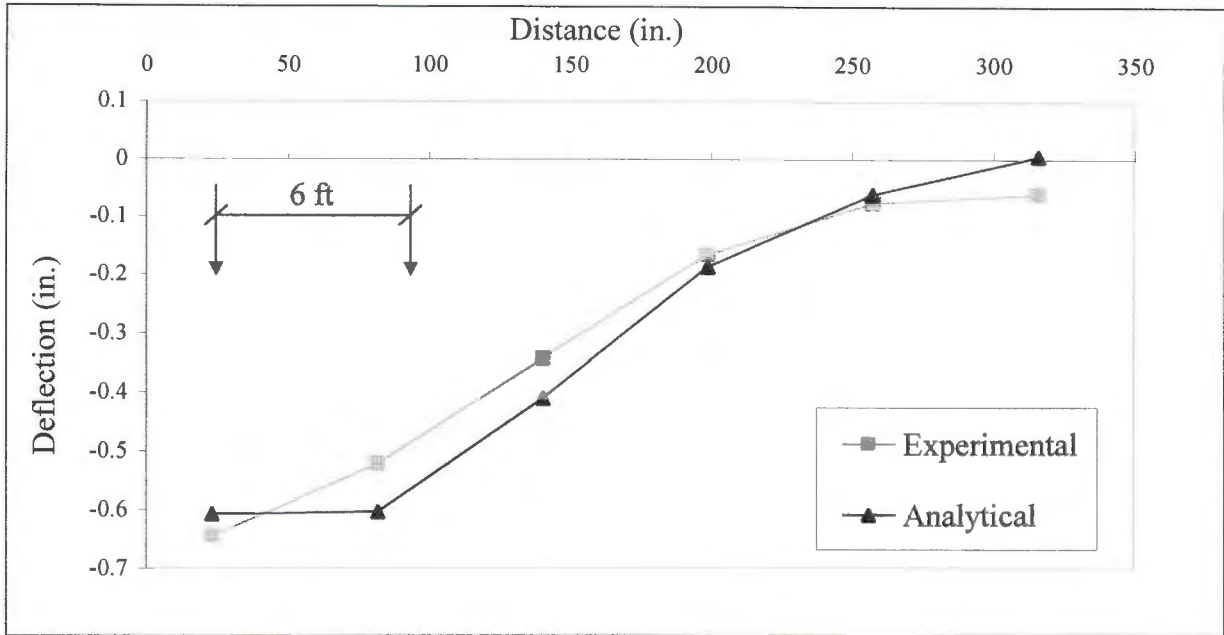
Fig. 5.46 Comparison of girder deflections at midspan for Load Case 5.
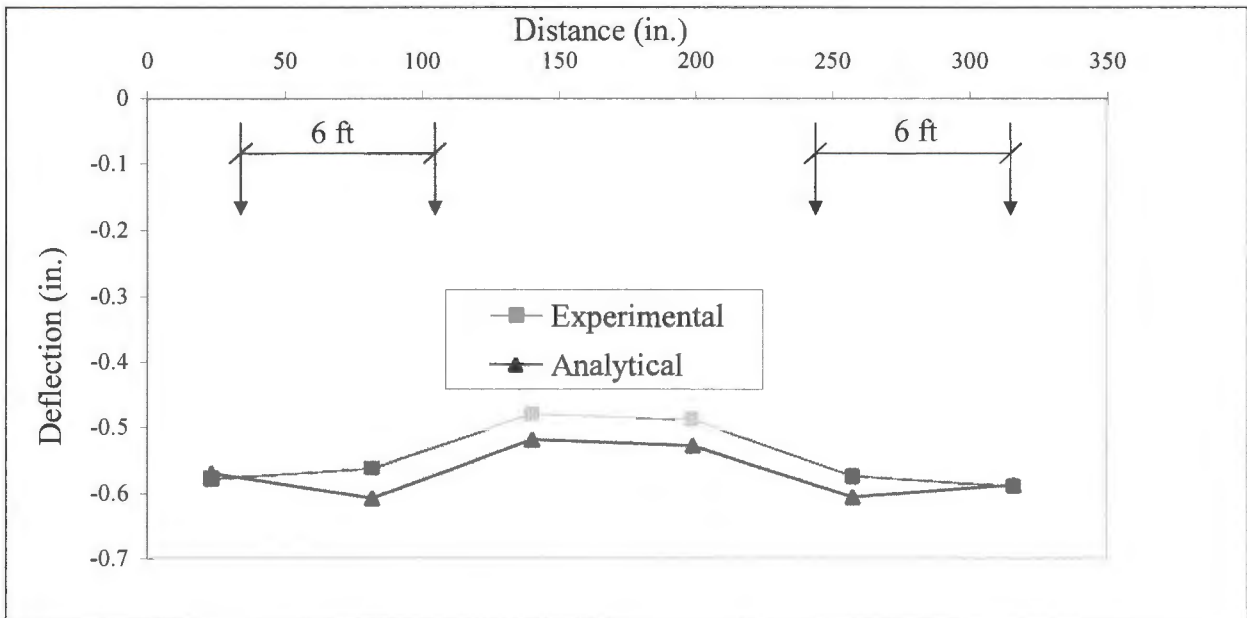
Fig. 5.47 Comparison of girder deflections at midspan for Load Case 6.

(a) Girder deflections at midspan.

(b) Girder bending stresses at midspan.
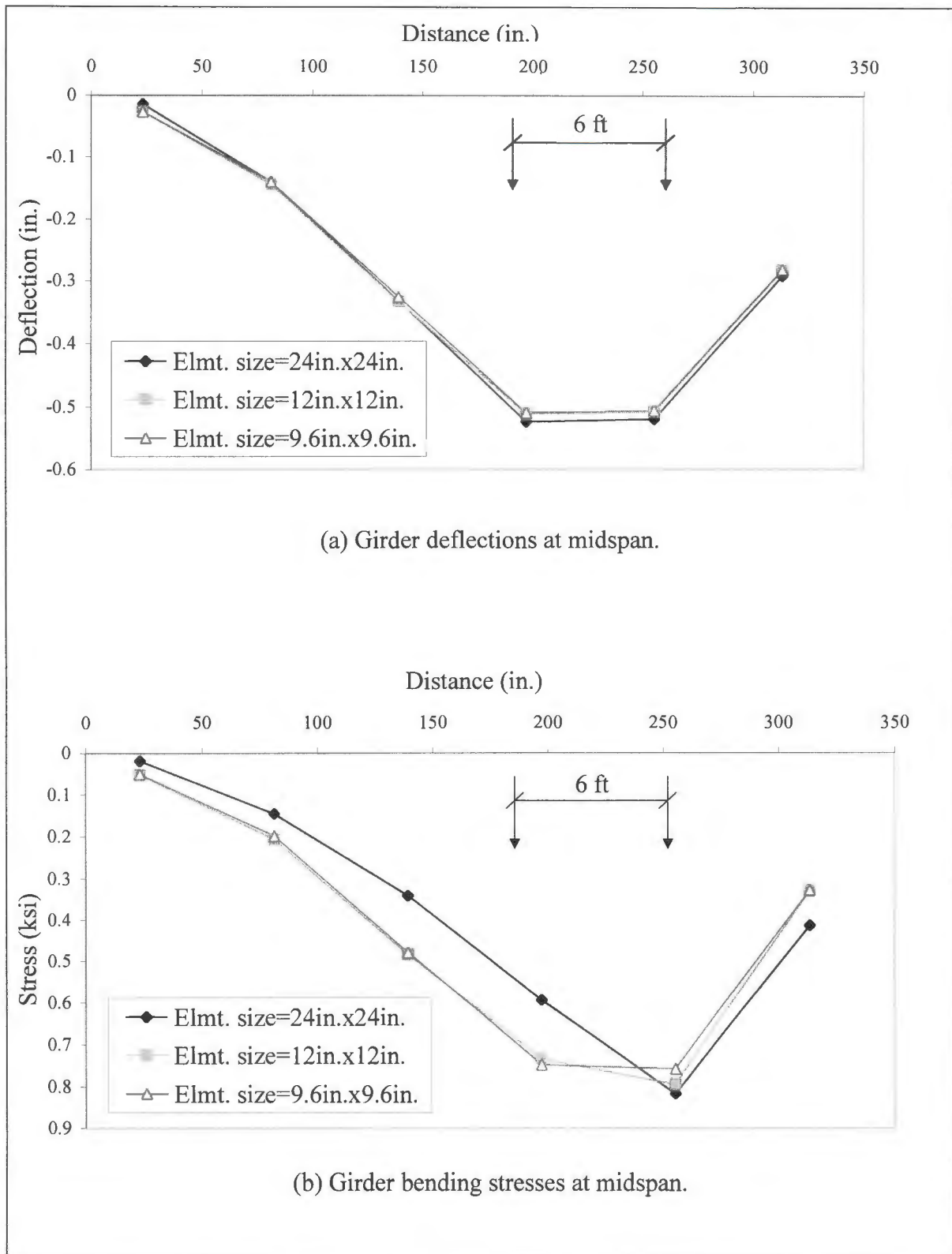
Fig. 5.48 Comparison of girder deflections and bending stresses at midspan for mesh sensitivity analysis of the Hibbard Creek Bridge.
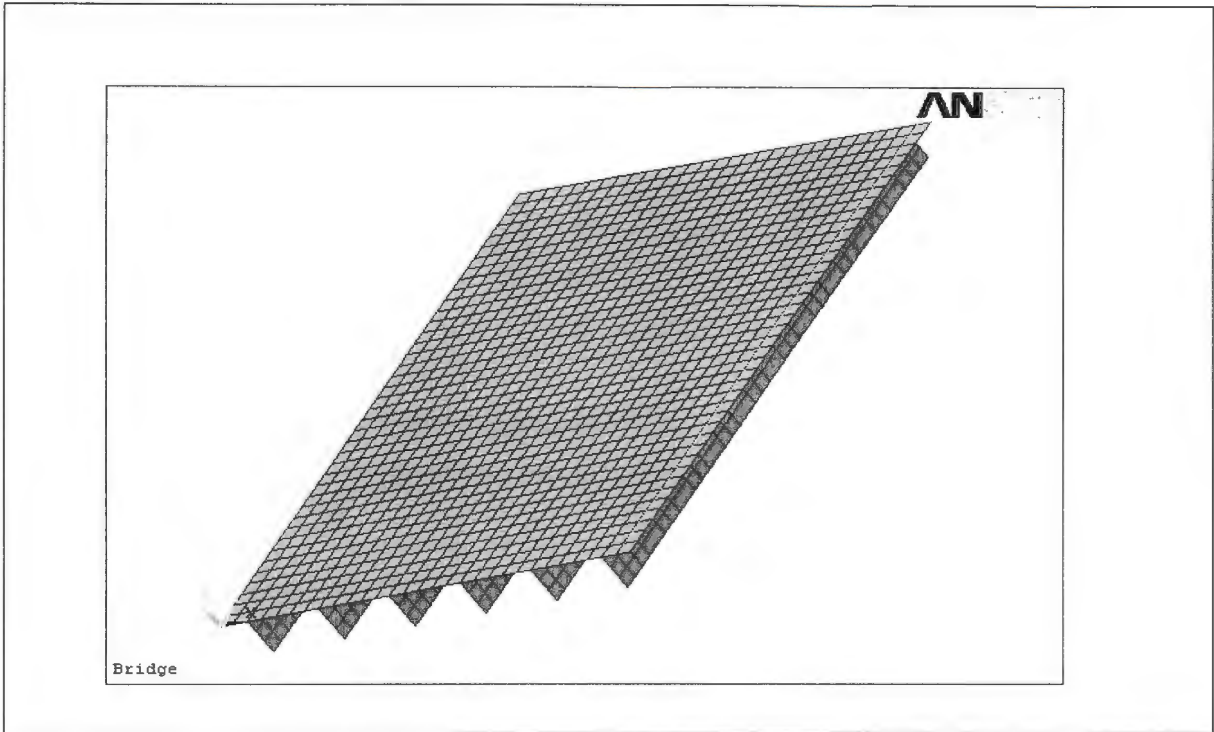
Fig. 5.49 Analytical model of the Cow Gulch Bridge.
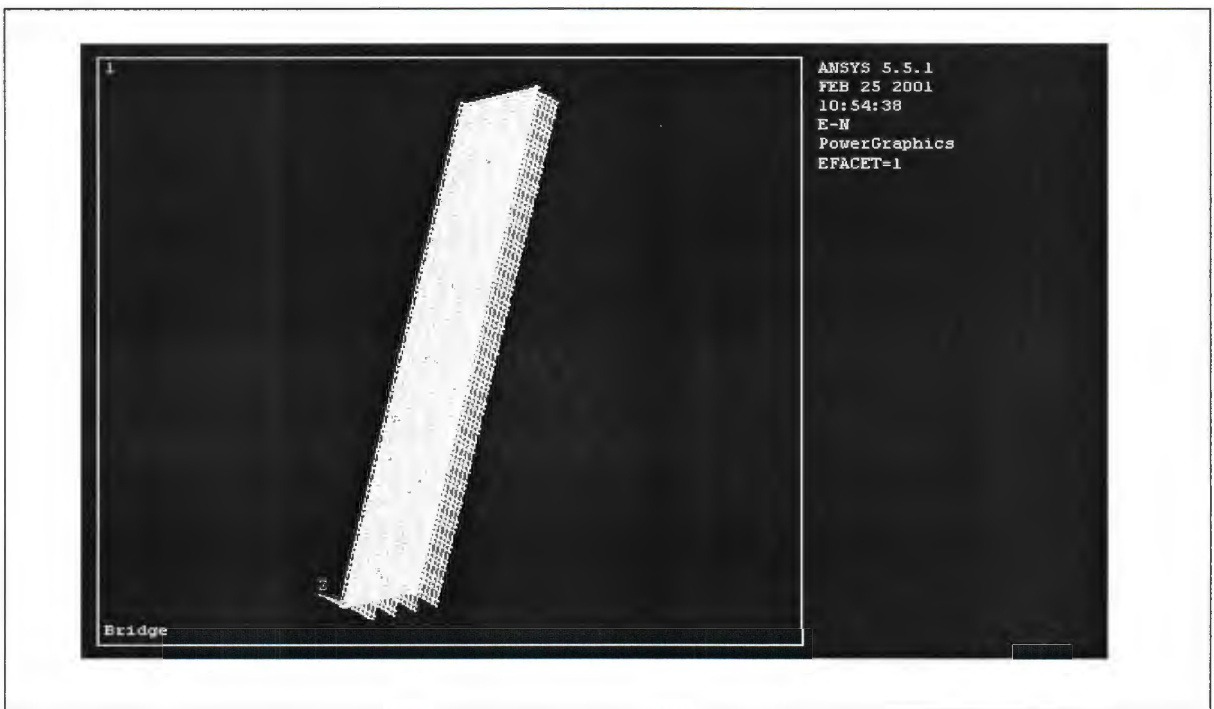
Fig. 5.50 Analytical model of the Wittson County Bridge – Span 3.
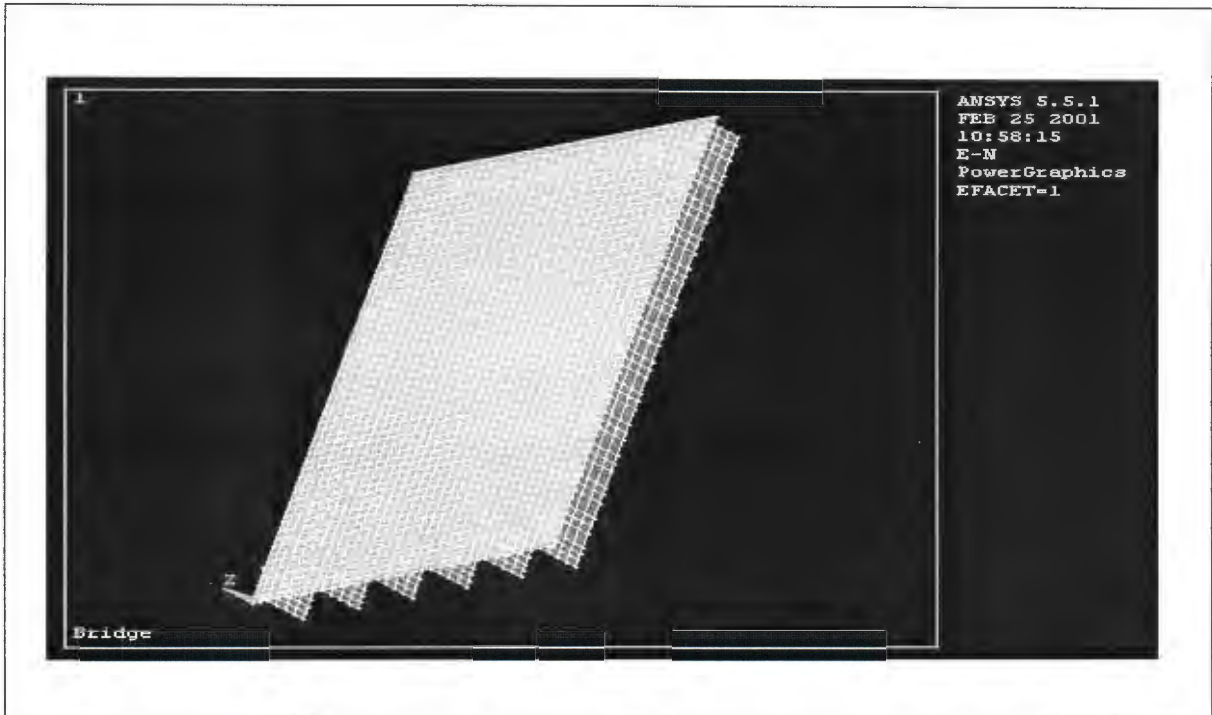
Fig. 5.51 Analytical model of the Chambers County Bridge.
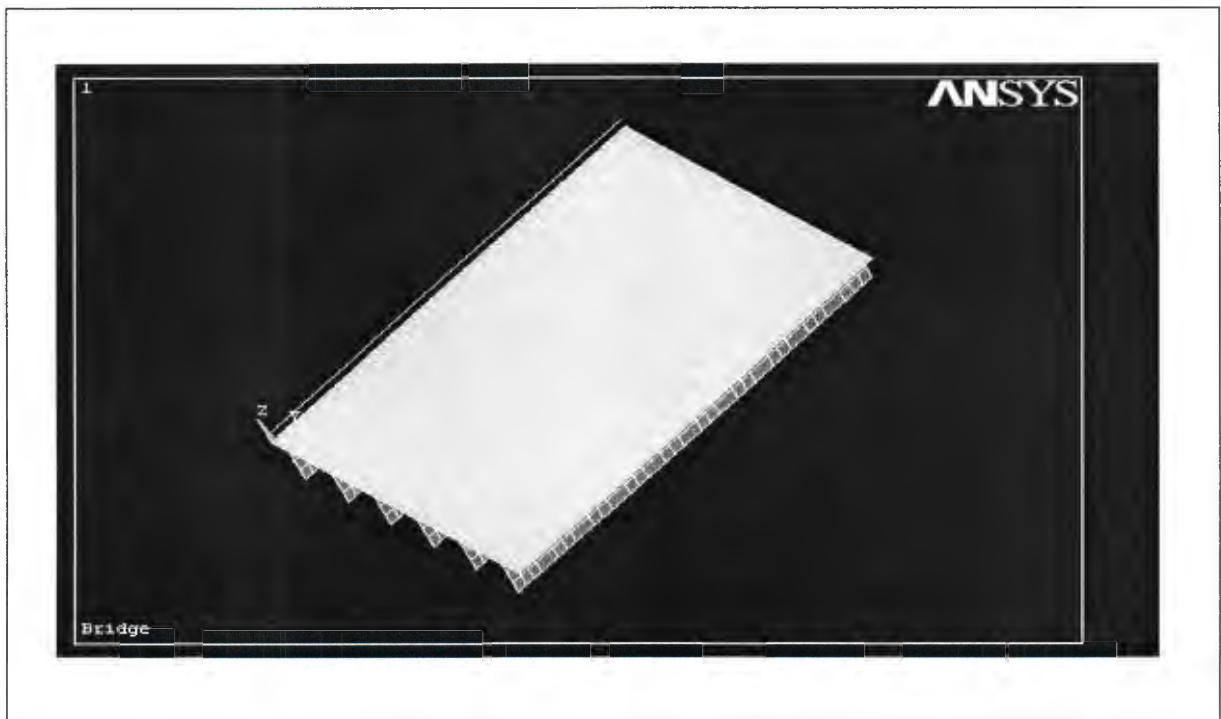
Fig. 5.52 Analytical model of the Hibbard Creek Bridge.

6. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary

Two programs were developed using the ANSYS Parametric Design Language (APDL), which is available in the ANSYS 5.5 general-purpose finite element software, to model and analyze glulam deck and girder bridges. In the program that modeled deck bridges, the deck panels were modeled with shell elements, and the stiffener beams and curbs were modeled with beam elements. In the program that modeled girder bridges, the deck panels and the girders were modeled with shell elements, while the curbs were modeled with beam elements.

In order to simplify the models of deck and girder bridges and for ease of computer coding, several assumptions were made:

1. The wearing surface was not modeled because it was assumed not to add any significant stiffness to the deck.

2. Wheel loads were assumed to be concentrated point loads since their tire footprints were assumed to be very small in comparison to the bridge dimensions.

3. The bridge was assumed to be simply supported and this was considered a conservative approach since in reality, there may be some rotational fixity at the abutments.

4. The guardrails and timber posts were not included in the model since this was not considered to affect the results significantly.

Experimental tests were performed on two deck bridges and four girder bridges by others and the observations were compared to the analytical results obtained using the programs developed in this study. Maximum design stresses of panels in the case of deck bridges and girders in the case of girder bridges were also compared to the results from the program.

6.2 Conclusions

From the results and discussions presented in this report, it can be concluded that the programs greatly simplify user input and reduce valuable modeling time required from the user for the finite element modeling and analysis of deck and girder bridges. From the parametric studies conducted on the bridges, it can be seen that the longitudinal modulus of elasticity of the deck in the case of deck bridges, the longitudinal modulus of elasticity of the girders in the case of girder bridges, and curb dimensions have significant effect on bridge response. In the case studies, the experimental deflections were generally bounded by the results obtained from the simply supported and the fixed conditions. This means that there could be some rotational restraint present on the abutments. Comparison of maximum design and analytical stresses obtained from the program indicate good correlation, and the user should always do this to check the results obtained from the program.

6.3 Recommendations

Based on the analytical modeling and the comparison of results, the following is recommended:

1. From the sensitivity study done on the bridges, the longitudinal modulus of elasticity of the deck in the case of deck bridges, longitudinal modulus of elasticity of the

girders in the case of girder bridges, and curb dimensions have significant effect on bridge response. The user should be careful when assessing the material properties of the bridge. Since the design handbook for wood [9] gives a range for each material property, and since the material properties of wood change with time, the user should consider doing a parametric study for the bridge, atleast for the properties mentioned above that have significant effect on bridge response.

2. Further study can be done to model the curbs and railings on the bridge. This would aid in better understanding the effect of edge stiffness due to the curbs and getting more realistic analytical results.

3. In the deck bridges, the stiffener beams have been assumed to be connected to the deck with rigid connections. The connectors, however, have different stiffness in tension and compression. A more appropriate study can be done to test the assumption made in this regard.

4. Currently, the programs analyze single-span bridges, but these programs can be extended to model and analyze multi-span bridges as well.

APPENDIX A. PREPROCESSOR LISTING FOR RIGHT ANGLED

LONGITUDINAL TIMBER DECK BRIDGE WITH COMMENTS

```
/filnam,Bridge
/title,Bridge
/prep7

! Enter preprocessor
! np = number of deck panels
! pw = width of deck panel
! pl = length of deck panel
! pt = thickness of deck panel (assumed constant throughout bridge)
! ns = number of stiffener beams
! bw = width of stiffener beam
! bt = height of stiffener beam
! exb = longitudinal modulus of elasticity of stiffener beams
! gb = shear modulus of stiffener beams
! The user is asked to input required data; the values below are default values.
*ask,np,number of deck panels,4
*ask,pw,panel width in inches,76.5
*ask,pl,panel length in inches,248.22
*ask,pt,panel thickness in inches,12
*ask,ns,number of stiffeners,1
*ask,bw,beam width in inches,6
*ask,bt,beam thickness in inches,12
*ask,exb,Modulus of elasticity of stiffener beams in psi,1800000
*ask,gb,Shear modulus of stiffener beams in psi,80000

! The panel is divided into 4 elements along the width of the panel
! div = number of elements along the traffic direction spanning the distance
! between the abutment and the closest stiffener beam
! Number of elements in the longitudinal direction = (div*(ns+1)) + 1
pw4=pw/4
le=pl/(ns+1)
n=le/pw4
*if,mod(nint(n),2),eq,0,then
        div=nint(n)
*else
        div=nint(n)-1
*endif
```

le1=le/div

! This segment generates nodes for the deck
n,1,0
ngen,5,1,all,,,pw4,
nmodif,2,6,
nmodif,4,pw-6,
ngen,np,5,all,,,pw,
ngen,(div*(ns+1))+1,np*5,all,,,,le1,

! Each panel is assigned a specific material property set consisting of longitudinal
! and transverse moduli of elasticity, and shear modulus
*ask,ans,Do all panels have same material properties (1 for yes/0 for no)?,1
*if,ans,eq,1,then
        *ask,eyp,Longitudinal E of panels in psi,1800000
        *ask,exp,Transverse E of panels in psi,100000
        *ask,gp,Shear modulus of panels in psi,130000

        et,1,shell63
        r,1,pt,
        *do,i,1,np,1
        uimp,i,EX,EY,,exp,eyp,
        uimp,i,NUXY,,,0.021,
        uimp,i,GXY,,,gp,
        mat,i
        en,1+((i-1)*4),(i*5)-4,(i*5)-3,(i*5)-3+(np*5),(i*5)-4+(np*5)
        egen,4,1,1+((i-1)*4),
        *enddo
        egen,div*(ns+1),np*5,all
*else
        *dim,eypanel,array,np
        *dim,expanel,array,np
        *dim,gpanel,array,np
        et,1,shell63
        r,1,pt
        *do,i,1,np,1
                *ask,eypanel(i),Longitudinal E of panel %i% in psi,2100000
                *ask,expanel(i),Transverse E of panel %i% in psi,124000
                *ask,gpanel(i),Shear modulus of panel %i% in psi,100000
                uimp,i,EX,EY,,expanel(i),eypanel(i),
                uimp,i,NUXY,,,0.021,
                uimp,i,GXY,,,gpanel(i),
                mat,i
                en,1+((i-1)*4),(i*5)-4,(i*5)-3,(i*5)-3+(np*5),(i*5)-4+(np*5)
                egen,4,1,1+((i-1)*4),

```
        *enddo
        egen,div*(ns+1),np*5,all
*endif

nsf=5*np*((div*(ns+1))+1)
esf=4*np*div*(ns+1)
```

! This routine selects the nodes on the deck to which the stiffener beam is connected
! and generates nodes for the stiffener beam(s).

```
nsel,S,NODE,,node(6,le,0)
nsel,A,NODE,,node(6,le,0)+2
*do,i,2,np,1,
nsel,A,NODE,,node(6,le,0)+((i-1)*5)
nsel,A,NODE,,node(pw-6,le,0)+((i-1)*5)
*enddo
*if,ns,gt,1,then
*do,i,2,ns,1
*do,j,1,np,1,
nsel,A,NODE,,node(6+(pw*(j-1)),le,0)+((np*5)*div*(i-1))
nsel,A,NODE,,node(pw-6+(pw*(j-1)),le,0)+((np*5)*div*(i-1))
*enddo
*enddo
*endif
ngen,2,nsf,all,,,,,-(pt+bt)/2
```

! Elements for the rigid connections between the deck and the stiffener beams are
! generated below.

```
nsel,all
et,np+1,beam4
r,np+1,100000,100000,100000,1,1,
uimp,np+1,EX, , ,100000,
et,np+2,beam4
r,np+2,100000,100000,100000,1,1,
uimp,np+2,EX, , ,100000,
type,np+1
real,np+1
mat,np+1
en,esf+1,node(6,le,0),nsf+node(6,le,0)
en,esf+2,node(pw-6,le,0),nsf+node(pw-6,le.0)
type,np+2
real,np+2
mat,np+2
en,esf+3,node(6,le,0),nsf+node(6,le,0)
en,esf+4,node(pw-6,le,0),nsf+node(pw-6,le,0)
egen,np,5,esf+1,esf+4,1
```

```
*if,ns,gt,1,then
        egen,ns,div*5*np,esf+1,esf+(4*np),1
*endif
```

! This segment generates elements for the stiffener beams.
```
esf1=esf+(4*np*ns)
et,np+3,beam4
keyopt,np+3,9,1
r,np+3,bw*bt,(bt*bw**3)/12,(bt**3*bw)/12,bt,bw,
rmore, ,bw*bt**3*(1/3-0.21*(bt/bw)*(1-((bt**4)/(12*bw**4))))
uimp,np+3,EX, , ,exb,
uimp,np+3,NUXY, , ,.37,
uimp,np+3,GXY, , ,gb,
type,np+3
real,np+3
mat,np+3
en,esf1+1,nsf+node(6,le,0),nsf+node(pw-6,le,0)
egen,np,5,esf1+1
*if,ns,gt,1,then
        egen,ns,div*5*np,esf1+1,esf1+np,1
*endif
en,esf1+(np*ns)+1,nsf+node(pw-6,le,0),nsf+node(6+pw,le,0)
egen,np-1,5,esf1+(np*ns)+1
*if,ns,gt,1,then
        egen,ns,div*5*np,esf1+(np*ns)+1,esf1+(np*ns)+(np-1),1
*endif
```

! Model curbs
! cw = width of curbs
! cd = depth of curbs
! Curbs are modeled as beam elements running along the edges of the bridge between
! abutments. The curbs are connected to the deck with rigid constraint equations.
```
*ask,curbs,Curbs(1 for yes,0 for no)?,1
*if,curbs,eq,1,then
*ask,cw,Enter the curb width,8
*ask,cd,Enter the curb depth,8
*get,emax1,ELEM,,num,max
*get,nmax1,NODE,,num,max
nsel,S,NODE,,1,1+(np*5*div*(ns+1)),np*5
nsel,A,NODE,,np*5,(np*5)+(np*5*div*(ns+1)),np*5
ngen,2,nmax1,all,,,,,(pt/2)+cd
nsel,all
r,np+4,cw*cd,(cd*cw**3)/12,(cd**3*cw)/12,cd,cw,
rmore, ,cw*cd**3*(1/3-0.21*(cd/cw)*(1-((cd**4)/(12*cw**4))))
real,np+4
```

```
en,emax1+1,1+nmax1,1+nmax1+(np*5)
en,emax1+2,(np*5)+nmax1,(np*5*2)+nmax1
egen,div*(ns+1),np*5,emax1+1,emax1+2,1
*do,i,1,1+(np*5*div*(ns+1)),np*5
cerig,i,i+nmax1,ALL
*enddo
*do,i,np*5,(np*5)+(np*5*div*(ns+1)),np*5
cerig,i,i+nmax1,ALL
*enddo
*endif


! Loads are modeled as either rectangular patch loads or concentrated loads depending
! upon the choice of the user. When the user chooses to model the loads as patch loads,
! the program calculates the equivalent concentrated load by multiplying the pressure
! due to the patch load to the patch area. If the patch load lies on two adjacent panels,
! the program calculates the area of patch load on each panel and calculates the
! equivalent concentrated loads.

*ask,loads,Patch loads (0) or Concentrated loads (1)?,1
*dim,X,array,50
*dim,Y,array,50
*dim,Q12,array,50
*if,loads,eq,1,then
        *ask,q,Enter the number of loads,1
        *do,i,1,q,1
                *ask,X(i),Enter X-coordinate of the load %i%
                *ask,Y(i),Enter Y-coordinate of the load %i%
                *ask,Q12(i),Enter the value of the load %i%,-9.1
        *enddo
*else
        *dim,X1,array,50
        *dim,Y1,array,50
        *dim,Q11,array,50
        *ask,q1,Number of patch loads?
        *do,i,1,q1,1
                *ask,X1((2*i)-1),Enter lower left X-coordinate of patch load %i%
                *ask,Y1((2*i)-1),Enter lower left Y-coordinate of patch load %i%
                *ask,X1(2*i),Enter upper right X-coordinate of patch load %i%
                *ask,Y1(2*i),Enter upper right Y-coordinate of patch load %i%
                *ask,Q11(i),Enter the pressure value of load %i%

                N1=X1((2*i)-1)/pw
                *if,nint(N1),ge,N1,then
                        N1=nint(N1)
                *else
```

```
                        N1=nint(N1)+1
                *endif

                N2=X1(2*i)/pw
                *if,nint(N2),ge,N2,then
                        N2=nint(N2)
                *else
                        N2=nint(N2)+1
                *endif

                *if,N1,eq,N2,then
                        X((2*i)-1)=(X1((2*i)-1) + X1(2*i))/2
                        Y((2*i)-1)=(Y1((2*i)-1) + Y1(2*i))/2
                        X(2*i)=0
                        Y(2*i)=0
                        Q12((2*i)-1)=Q11(i)*(X1(2*i)-X1((2*i)-1))*(Y1(2*i)-Y1((2*i)-1))
                        Q12(2*i)=0
                *else
                        X((2*i)-1)=((N1*pw)+X1((2*i)-1))/2
                        Y((2*i)-1)=(Y1((2*i)-1)+Y1(2*i))/2
                        X(2*i)=((N1*pw)+X1(2*i))/2
                        Y(2*i)=Y((2*i)-1)
                        Q12((2*i)-1)=Q11(i)*((N1*pw)-X1((2*i)-1))*(Y1(2*i)-Y1((2*i)-1))
                        Q12(2*i)=Q11(i)*(X1(2*i)-(N1*pw))*(Y1(2*i)-Y1((2*i)-1))
                *endif
        *enddo
        q=2*q1
*endif


! This segment calculates the net concentrated force acting on each node. This becomes
! necessary when for example, during distribution of loads to the nodes of the
! rectangular elements, some nodes may have more than one load acting on it. The loads
! are distributed to the corners of the rectangular elements using Desai's [11]
! interpolation functions as explained in Chapter 2.
*dim,count,array,50
*dim,foz,array,50
*dim,mox,array,50
*dim,moy,array,50

*do,i,1,q,1
        Xo=X(i)
        Yo=Y(i)
        F=Q12(i)

        t=Xo/pw
```

```
*if,nint(t),gt,t,then
        t=nint(t)
*else
        t=nint(t)+1
*endif

*if,Xo-((t-1)*pw),lt,6,then
        nt=1
*elseif,Xo-((t-1)*pw),lt,pw/2,then
        nt=2
*elseif,Xo-((t-1)*pw),le,pw-6,then
        nt=3
*else
        nt=4
*endif

p=Yo/le1
*if,nint(p),gt,p,then
        p=nint(p)
*else
        p=nint(p)+1
*endif

node1=(t*5)-4+(nt-1)+((np*5)*(p-1))
node2=(t*5)-4+nt+((np*5)*(p-1))
node3=node2+(np*5)
node4=node1+(np*5)

xe=Xo-nx(node1)
ye=Yo-ny(node1)
st=xe/(nx(node2)-nx(node1))
tt=ye/le1
nx1=1-(3*(st**2))+(2*(st**3))
nx2=(st**2)*(3-(2*st))
nx3=(nx(node2)-nx(node1))*st*((st-1)**2)
nx4=(nx(node2)-nx(node1))*(st**2)*(st-1)

ny1=1-(3*(tt**2))+(2*(tt**3))
ny2=(tt**2)*(3-(2*tt))
 ny3=le1*tt*((tt-1)**2)
ny4=le1*(tt**2)*(tt-1)

f1=nx1*ny1*F
my1=(-1)*nx3*ny1*F
mx1=nx1*ny3*F
```

```
f2=nx2*ny1*F
my2=(-1)*nx4*ny1*F
mx2=nx2*ny3*F

f3=nx2*ny2*F
my3=(-1)*nx4*ny2*F
mx3=nx2*ny4*F

f4=nx1*ny2*F
my4=(-1)*nx3*ny2*F
mx4=nx1*ny4*F

*do,t,1,4*i,1
        *if,count(t),ne,node1,then
                *if,count(t),eq,0,then
                        dot=0
                        *do,z,1,t,1
                                *if,count(z),eq,node1,then
                                        dot=dot+1
                                *endif
                        *enddo
                        *if,dot,eq,0,then
                                count(t)=node1
                                foz(t)=f1
                                mox(t)=mx1
                                moy(t)=my1
                        *endif
                *endif
        *else
                foz(t)=foz(t)+f1
                mox(t)=mox(t)+mx1
                moy(t)=moy(t)+my1

        *endif

        *if,count(t),ne,node2,then
                *if,count(t),eq,0,then
                        dot=0
                        *do,z,1,t,1
                                *if,count(z),eq,node2,then
                                        dot=dot+1
                                *endif
                        *enddo
                        *if,dot,eq,0,then
```

```
                                count(t)=node2
                                foz(t)=f2
                                mox(t)=mx2
                                moy(t)=my2
                        *endif
                *endif
        *else
                foz(t)=foz(t)+f2
                mox(t)=mox(t)+mx2
                moy(t)=moy(t)+my2
        *endif

        *if,count(t),ne,node3,then
                *if,count(t),eq,0,then
                        dot=0
                        *do,z,1,t,1
                                *if,count(z),eq,node3,then
                                        dot=dot+1
                                *endif
                        *enddo
                        *if,dot,eq,0,then
                                count(t)=node3
                                foz(t)=f3
                                mox(t)=mx3
                                moy(t)=my3
                        *endif
                *endif
        *else
                foz(t)=foz(t)+f3
                mox(t)=mox(t)+mx3
                moy(t)=moy(t)+my3
        *endif

        *if,count(t),ne,node4,then
                *if,count(t),eq,0,then
                        dot=0
                        *do,z,1,t,1
                                *if,count(z),eq,node4,then
                                        dot=dot+1
                                *endif
                        *enddo
                        *if,dot,eq,0,then
                                count(t)=node4
                                foz(t)=f4
                                mox(t)=mx4
```

```
                              moy(t)=my4
                        *endif
                  *endif
            *else
                  foz(t)=foz(t)+f4
                  mox(t)=mox(t)+mx4
                  moy(t)=moy(t)+my4
            *endif
      *enddo
*enddo

num=0
*do,L,1,50,1
      *if,count(L),ne,0,then
            num=num+1
      *endif
*enddo

finish
/solu
```

! This segment applies the loads on the nodes of the rectangular elements.
```
*do,t,1,num,1
      f,count(t),FZ,foz(t)
      f,count(t),MX,mox(t)
      f,count(t),MY,moy(t)
*enddo
```

! Displacement constraints for the abutments. The bridge is assumed to be simply
! supported.
```
d,1,UX,,,np*5,1,UY,UZ,ROTY,ROTZ
d,1+(np*5*div*(ns+1)),UY,,,(np*5)+(np*5*div*(ns+1)),1,UZ,ROTY,ROTZ

/pbc,all,,1
eplot
```

! If user is satisfied with model, enter solution=1
```
*ask,solution, Proceed with solution?(1 for yes, 0 for no),1
*if,solution,eq,1,then
```
! Solve the system for the given loading.
```
/solu
solve
finish
```

! Enter post processor module. The output from the program is redirected to the file

! "Results.dat" which resides in the current home directory of the user. Nodes of the
! panels at midspan are selected and the midspan deflections are printed in the output.

```
/post1
plnsol,U,Z,0,1
*dim,nodenum,array,2*np
*dim,nodenumX,array,2*np
*dim,nodenumY,array,2*np
*dim,nodedisp,array,2*np
*dim,Label,char,1,3
*do,i,1,np,1
        nodenum((2*i)-1)=NODE((i-1)*pw,pl/2,0)
        nodenumX((2*i)-1)=NX(nodenum((2*i)-1))
        nodenumY((2*i)-1)=NY(nodenum((2*i)-1))
        nodenum(2*i)=NODE(i*pw,pl/2,0)
        nodenumX(2*i)=NX(nodenum(2*i))
        nodenumY(2*i)=NY(nodenum(2*i))
*enddo
*do,i,1,2*np,1
        *if,i,gt,1,then
                *if,nodenum(i),eq,nodenum(i-1),then
                        nodenum(i)=nodenum(i)+1
                *endif
        *endif
        nodedisp(i)=UZ(nodenum(i))
*enddo
Label(1,1)='X-coord'
Label(1,2)='Y-coord'
Label(1,3)='UZ'
/out,Results,dat,,
/com,Midspan Panel deflections (in inches)
/com,-----------------------------------
/com
*vwrite,Label(1,1),Label(1,2),Label(1,3)
(A10,A10,A10)
*vwrite,nodenumX(1),nodenumY(1),nodedisp(1)
(F8.2,' ',F8.2,' ',F8.4)
/out
```

! The user is given a choice to pick the panels for which the user requires stress results.
! The stress results can be either for all the elements in the panels or just the maximum
! values. This again is the choice of the user. The stress results are calculated at the
! centroid of each element at mid-thickness.

```
*dim,panelnum,array,np+1
*do,T,1,np+1,1
        *ask,panelnum(T),Enter panel no. for stress results (0 to stop),
```

```
            *if,panelnum(T),eq,0,then
                    *exit
            *endif
    *enddo
    *if,panelnum(1),ne,0,then
    esel,S,MAT,,panelnum(1)
    *do,i,1,T-1,
    esel,A,MAT,,panelnum(i)
    *enddo
    plnsol,u,z
    *endif

    *dim,elemnum,array,4*div*(ns+1)
    *dim,stressX,array,4*div*(ns+1)
    *dim,stressY,array,4*div*(ns+1)
    *dim,stressXY,array,4*div*(ns+1)
    *dim,elemnumX,array,4*div*(ns+1)
    *dim,elemnumY,array,4*div*(ns+1)
    *dim,Label1,char,1,4
    *if,panelnum(1),ne,0,then
    *ask,outp,Do you want the complete listing? (1 for yes/0 for no),0
    *do,L,1,T-1,1
            panlab='Panel %panelnum(L)%'
            /out,Results,dat,,append
    /com,
    /com,
    *vwrite,panlab
    (A8)
            /out
            panel=panelnum(L)
            *do,i,1,div*(ns+1),1
                    elemnum((4*i)-3)=((panel-1)*4)+1+(np*4*(i-1))
                    elemnumX((4*i)-3)=centrx(elemnum((4*i)-3))
                    elemnumY((4*i)-3)=centry(elemnum((4*i)-3))
                    elemnum((4*i)-2)=elemnum((4*i)-3)+1
                    elemnumX((4*i)-2)=centrx(elemnum((4*i)-2))
                    elemnumY((4*i)-2)=centry(elemnum((4*i)-2))
                    elemnum((4*i)-1)=elemnum((4*i)-3)+2
                    elemnumX((4*i)-1)=centrx(elemnum((4*i)-1))
                    elemnumY((4*i)-1)=centry(elemnum((4*i)-1))
                    elemnum(4*i)=elemnum((4*i)-3)+3
                    elemnumX(4*i)=centrx(elemnum(4*i))
                    elemnumY(4*i)=centry(elemnum(4*i))
            *enddo
            etable,strX,s,x
```

```
        etable,strY,s,y
        etable,strXY,s,xy
        *do,i,1,4*div*(ns+1)
                *get,stressX(i),ELEM,elemnum(i),ETAB,strX,
                *get,stressY(i),ELEM,elemnum(i),ETAB,strY,
                *get,stressXY(i),ELEM,elemnum(i),ETAB,strXY,
        *enddo

*if,outp,eq,0,then
MaxstrX=abs(stressX(1))
MaxstrY=abs(stressY(1))
MaxstrXY=abs(stressXY(1))
*do,r,1,4*div*(ns+1),1
*if,abs(stressX(r)),ge, MaxstrX,then
MaxstrX=abs(stressX(r))
maxnodX1=elemnumX(r)
maxnodY1=elemnumY(r)
*endif
*enddo

*do,r,1,4*div*(ns+1),1
*if,abs(stressY(r)),ge, MaxstrY,then
MaxstrY=abs(stressY(r))
maxnodX2=elemnumX(r)
maxnodY2=elemnumY(r)
*endif
*enddo

*do,r,1,4*div*(ns+1),1
*if,abs(stressXY(r)),ge, MaxstrXY,then
MaxstrXY=abs(stressXY(r))
maxnodX3=elemnumX(r)
maxnodY3=elemnumY(r)
*endif
*enddo
/out,Results,dat,,append
*vwrite,MaxstrY
('Maximum longitudinal stress in panel is '.F6.4,' ksi.')
/com,At location
*vwrite,maxnodX2
('                X = ',F7.2,' in.')
*vwrite,maxnodY2
('                Y = ',F7.2,' in.')
/com,
*vwrite,MaxstrX
```

```
('Maximum tranverse stress in panel is ',F6.4,' ksi.')
/com,At location
*vwrite,maxnodX1
('              X = ',F7.2,' in.')
*vwrite,maxnodY1
('              Y = ',F7.2,' in.')
/com,
*vwrite,MaxstrXY
('Maximum shear stress in panel is ',F6.4,' ksi.')
/com,At location
*vwrite,maxnodX3
('              X = ',F7.2,' in.')
*vwrite,maxnodY3
('              Y = ',F7.2,' in.')
/out
*else
        Label1(1,1)='Stress-Long'
        Label1(1,2)='Stress-Transverse'
        Label1(1,3)='Shear stress'
        /out,Results,dat,,append
        /com,
        /com,Stresses in desired panel (in ksi)
        /com,---------------------------------
*vwrite,Label(1,1),Label(1,2),Label1(1,1),Label1(1,2),Label1(1,3)
(A10,A10,A12,A12,A12)
*vwrite,elemnumX(1),elemnumY(1),stressY(1),stressX(1),stressXY(1)
(F8.2,' ',F8.2,' ',F12.4,' ',F12.4,' ',F12.4)
        /out
*endif
*enddo
*endif


! The stiffener beam stress results are also redirected to the output file. In case of even
! number of stiffener beams, the stress results are listed for the stiffener beam closest to
! midspan. In case of odd number of stiffener beams, the stress results are listed for the
! stiffener beam at midspan. The stress results listed are bending stresses at the bottom
! of the stiffener beam.
esel,all
*dim,beamelem,array,2*np-1
*dim,beamelmX,array,2*np-1
*dim,beamelmY,array,2*np-1
*dim,beamstr,array,2*np-1
*if,mod(ns,2),ne,0,then
        /out,Results,dat,,append
        /com,
```

```
/com,Stresses in stiffener beam at midspan (in ksi)
/com,-------------------------------------------
/out
*do,i,1,np-1,1
        beamelem((2*i)-1)=esf1+((nint(ns/2)-1)*np)+i
        beamelmX((2*i)-1)=centrx(beamelem((2*i)-1))
        beamelmY((2*i)-1)=centry(beamelem((2*i)-1))
        beamelem(2*i)=esf1+(np*ns)+((nint(ns/2)-1)*np)+(i-1)
        beamelmX(2*i)=centrx(beamelem(2*i))
        beamelmY(2*i)=centry(beamelem(2*i))
*enddo
beamelem((2*np)-1)=esf1+((nint(ns/2)-1)*np)+np
beamelmX((2*np)-1)=centrx(beamelem((2*np)-1))
beamelmY((2*np)-1)=centry(beamelem((2*np)-1))
*else
/out,Results,dat,,append
/com,
/com,Stresses in stiffener beam close to midspan (in ksi)
/com,-----------------------------------------------------
/out
*do,i,1,np-1,1
        beamelem((2*i)-1)=esf1+(nint(ns/2)*np)+i
        beamelmX((2*i)-1)=centrx(beamelem((2*i)-1))
        beamelmY((2*i)-1)=centry(beamelem((2*i)-1))
        beamelem(2*i)=esf1+(np*ns)+(nint(ns/2)*np)+(i-2)
        beamelmX(2*i)=centrx(beamelem(2*i))
        beamelmY(2*i)=centry(beamelem(2*i))
*enddo
beamelem((2*np)-1)=esf1+(nint(ns/2)*np)+np
beamelmX((2*np)-1)=centrx(beamelem((2*np)-1))
beamelmY((2*np)-1)=centry(beamelem((2*np)-1))
*endif
ETABLE,bstr,LS,10
*do,i,1,(2*np)-1,1
        *get,beamstr(i),ELEM,beamelem(i),ETAB,bstr,
*enddo
Label1(1,4)='Bend. Stress'
/out,Results,dat,,append
/com,
*vwrite,Label(1,1),Label(1,2),Label1(1,4)
(A10,A10,A10)
*vwrite,beamelmX(1),beamelmY(1),beamstr(1)
(F8.2,' ',F8.2,' ',F12.4)
/out
*endif
```

APPENDIX B. PREPROCESSOR LISTING FOR TIMBER GLULAM GIRDER

BRIDGE WITH COMMENTS

```
/filnam,Bridge
/title,Bridge
/prep7

! Enter preprocessor
! theta = skew angle of bridge in degrees (negative if left end is ahead)
! sb = span of bridge in inches
! pw = full panel width in inches
! pl = panel length along skew in inches
! pt = panel thickness in inches
! ns = number of girders
! bw = girder width
! bt = girder height
! ld = left overhang
! rd = right overhang
! bpwidth = width of partial panel at the bottom of model (defaults to zero)
! tpwidth = width of partial panel at the top of model (defaults to zero)
! np = number of full panels in the deck
! elsize = user selected element size in inches
*ask,theta,angle of skew in degrees (-ve if left end is ahead),-30
*ask,sb,span of bridge in inches,472.92
*ask,pw,full panel width in inches,48
*ask,pl,panel length in inches along theta,388.8
*ask,pt,panel thickness in inches,5.125
*ask,ns,number of girders,6
*ask,bw,girder width in inches,8.75
*ask,bt,girder thickness in inches,28.5
*ask,ld,left overhang in inches,23.375
*ask,rd,right overhang in inches,23.5
*ask,ppanel,partial panels near the abutments?(0 for no,1 for yes),1
pi=3.14
*if,ppanel,eq,1,then
*ask,bpwidth,width of partial panel at the bottom,40.92
*ask,tpwidth,width of partial panel at the top,0
*else
bpwidth=0
tpwidth=0
*endif
np=nint((sb-bpwidth-tpwidth)/pw)
```

```
sd=((pl*cos(theta*pi/180))-rd-ld)/(ns-1)
*ask,elsize,Element size in inches?,12
pw4=elsize

num1=ld/(pw4*cos(theta*pi/180))
*if,nint(num1),lt,num1,then
num1=nint(num1)+1
*else
num1=nint(num1)
*endif
```

! This segment generates nodes for the deck

```
n,1,0,bpwidth,
local,11,0,0,0,0,theta
ngen,num1+1,1,all,,,ld/(num1*cos(theta*pi/180))

num2=sd/(pw4*cos(theta*pi/180))
*if,nint(num2),lt,num2,then
num2=nint(num2)+1
*else
num2=nint(num2)
*endif

*if,ns-1,ne,0,then
ngen,(num2*(ns-1))+1,1,num1+1,,,sd/(num2*cos(theta*pi/180))
*endif

num3=rd/(pw4*cos(theta*pi/180))
*if,nint(num3),lt,num3,then
num3=nint(num3)+1
*else
num3=nint(num3)
*endif

ngen,num3+1,1,num1+(num2*(ns-1))+1,,,rd/(num3*cos(theta*pi/180))

diff=num1+(num2*(ns-1))+num3+1
temp=pw/pw4
*if,nint(temp),lt,temp,then
temp=nint(temp)+1
*else
temp=nint(temp)
*endif

csys,0
```

```
ngen,temp+1,diff,all,,,,pw/temp
ngen,np,diff*(temp+1),all,,,,pw

*get,botmax,NODE,,num,max
```

! **Nodes for partial panels**
! ---
```
*if,bpwidth,gt,0,then
ngen,2,botmax,1,diff,1,,-bpwidth
p1=bpwidth/pw4
*if,nint(p1),lt,p1,then
p1=nint(p1)+1
*else
p1=nint(p1)
*endif
*if,p1,ge,1,then
ngen,p1+1,diff,botmax+1,botmax+diff,1,,(bpwidth/p1)
*else
ngen,2,diff,botmax+1,botmax+diff,1,,bpwidth
*endif
*endif


*get,topmax,NODE,,num,max
*if,tpwidth,gt,0,then
ngen,2,topmax,botmax,botmax-diff+1,-1,,
c1=tpwidth/pw4
*if,nint(c1),lt,c1,then
c1=nint(c1)+1
*else
c1=nint(c1)
*endif
*if,c1,ge,1,then
ngen,c1+1,diff,botmax+topmax,botmax+topmax-diff+1,-1,,tpwidth/c1
*else
ngen,2,diff,botmax+topmax,botmax+topmax-diff+1,-1,,tpwidth
*endif
*endif
```

! **Deck element properties**
!---------------------------------
! **Each panel is assigned a specific material property set consisting of longitudinal**
! **and transverse moduli of elasticity, and shear modulus. After the user inputs the**
! **properties, elements are generated on the deck**
```
*ask,ans,Do all panels have same material properties (1 for yes/0 for no)?,1
*if,ans,eq,1,then
        *ask,exp,Longitudinal E of panels in psi,1800000
```

```
        *ask,eyp,Transverse E of panels in psi,130000
        *ask,gp,Shear modulus of panels in psi,100000

        et,1,shell63
        r,1,pt,
        *do,i,1,np,1
        uimp,i,EX,EY,,exp,eyp,
        uimp,i,NUXY,,,0.021,
        uimp,i,GXY,,,gp,
        mat,i
*enddo
                mat,1,
                en,1,1,2,2+diff,1+diff
                egen,diff-1,1,1,
                egen,temp,diff,all
                egen,np,(temp+1)*diff,all,,,1
*else
        *dim,eypanel,array,np
        *dim,expanel,array,np
        *dim,gpanel,array,np
        et,1,shell63
        r,1,pt
        *do,i,1,np,1
                *ask,expanel(i),Longitudinal E of full panel %i% in psi,1800000
                *ask,eypanel(i),Transverse E of full panel %i% in psi,130000
                *ask,gpanel(i),Shear modulus of full panel %i% in psi,100000
                uimp,i,EX,EY,,expanel(i),eypanel(i),
                uimp,i,NUXY,,,0.021,
                uimp,i,GXY,,,gpanel(i),
                mat,i
*enddo
                mat,1,
                en,1,1,2,2+diff,1+diff
                egen,diff-1,1,1,
                egen,temp,diff,all
                egen,np,(temp+1)*diff,all,,,1
*endif

! Lower partial panel elements
! ----------------------------
*get,elmax1,ELEM,,num,max
*if,bpwidth,gt,0,then
        mat,1
        egen,2,botmax,1,diff-1,1,
        *if,p1,gt,1,then
```

```
egen,p1,diff,elmax1+1,elmax1+diff,1,
        *endif
*endif
```

! Upper partial panel elements
! ----------------------------
```
*get,elmax2,ELEM,,num,max
*if,tpwidth,gt,0,then
        mat,np
        egen,2,topmax+diff,elmax1,elmax1-diff+2,-1,
        *if,c1,gt,1,then
        egen,c1,diff,elmax2+1,elmax2+diff,1,
        *endif
*endif
```

! Girder nodes
! --------------
! This routine selects the nodes on the deck to which the girders are connected and
! generates nodes for the girders
```
*get,nmax1,node,,num,max
*if,bpwidth,gt,0,then
nsel,S,node,,node(ld,ld*tan(theta*pi/180),0)
*do,j,0,ns-1,1,
*if,p1,gt,1,then
*do,k,0,p1-1,1
nsel,A,node,,node(ld,ld*tan(theta*pi/180),0)+(num2*j)+(diff*k)
*enddo
*else
nsel,A,node,,node(ld,ld*tan(theta*pi/180),0)+(num2*j)
*endif
*enddo
*endif

*if,tpwidth,gt,0,then
*if,bpwidth,eq,0,then
nsel,S,node,,node(ld,sb+(ld*tan(theta*pi/180)),0)
*else
nsel,A,node,,node(ld,sb+(ld*tan(theta*pi/180)),0)
*endif
*do,jt,0,ns-1,1,
*if,c1,gt,1,then
*do,k,0,c1-1,1
nsel,A,node,,node(ld,sb+(ld*tan(theta*pi/180)),0)+(num2*jt)-(diff*k)
*enddo
*else
```

```
nsel,A,node,,node(ld,ld*tan(theta*pi/180),0)+(num2*jt)
*endif
*enddo
*endif

*if,bpwidth,eq,0,then
*if,tpwidth,eq,0,then
nsel,S,node,,node(ld,ld*tan(theta*pi/180),0)
*else
nsel,A,node,,num1+1
*endif
*do,j,1,ns-1,1,
nsel,A,node,,num1+1+(num2*j)
*enddo
*endif

*if,tpwidth,eq,0,then
*if,bpwidth,eq,0,then
*do,jt,0,ns-1,1
nsel,A,node,,botmax-diff+1+num1+(num2*jt)
*enddo
*else
*do,jt,0,ns-1,1
nsel,A,node,,botmax-diff+1+num1+(num2*jt)
*enddo
*endif
*endif

*do,i,1,np,1,
*do,jp,0,ns-1,1
*do,k,1,temp-1,1,
nsel,A,node,,((i-1)*(temp+1)*diff)+1+num1+(num2*jp)+(diff*k)
*enddo
*enddo
*enddo

temp1=(bt+(pt/2))/pw4
*if,nint(temp1),lt,temp1,then
temp1=nint(temp1)+1
*else
temp1=nint(temp1)
*endif
```

! Once the common nodes between the girders and deck are selected, nodes are
! generated along the height of the girder

```
ngen,temp1+1,nmax1,all,,,,,-((bt+(pt/2))/temp1)
```

! Girder elements
! --------------------
! Properties for the girders are obtained from the user and the girder elements are
! generated. The program also allows the user to provide different material properties
! for each girder

```
*get,emax1,elem,,num,max
et,2,shell63
r,2,bw,
real,2,
type,2,
*ask,ans,Do all girders have same material properties (1 for yes,0 for no),1
*if,ans,eq,1,then
*ask,exb,Longitudinal modulus of elasticity of girders in psi,2000000
*ask,eyb,Transverse modulus of elasticity of girders in psi,240000
*ask,gb,Shear modulus of girders in psi,106000
*do,jt,1,ns,1
uimp,np+jt,EX,EY, ,exb,eyb
uimp,np+jt,NUXY, , ,0.021,
uimp,np+jt,GXY, , ,gb,
*enddo
*else
*dim,eystringer,array,ns
*dim,exstringer,array,ns
*dim,gstringer,array,np
*do,jt,1,ns,1
                *ask,eystring(jt),Longitudinal E of girder %jt% in psi,2020000
                *ask,exstring(jt),Transverse E of girder %jt% in psi,240000
                *ask,gstring(jt),Shear modulus of girder %jt% in psi,106000
                uimp,jt,EX,EY,,estring(jt),eystring(jt),
                uimp,jt,NUXY,,,0.021,
                uimp,jt,GXY,,,gstring(jt),
*enddo
*endif

*do,i,1,np,1,
*do,j,0,ns-1,1,
mat,np+j+1
z=((i-1)*diff*(temp+1))+diff+(num2*j)
e,z+1+num1,z+1+num1+diff,z+1+num1+diff+nmax1,z+1+num1+nmax1
*get,emaxi,elem,,num,max
egen,temp-2,diff,emaxi,
*enddo
*enddo
```

```
*do,i,1,np-1,1,
*do,j,0,ns-1,1,
z1=(i*diff*(temp+1))-(diff*2)+(num2*j)
z2=(i*diff*(temp+1))+diff+(num2*j)
e,z1+1+num1,z2+1+num1,z2+1+num1+nmax1,z1+1+num1+nmax1
*enddo
*enddo
```

! Generates elements on the girders if a bottom partial panel exists on the deck
```
*if,bpwidth,eq,0,then
*do,j,0,ns-1,1
z=1+num1+(num2*j)
mat,np+j+1
e,z,z+diff,z+diff+nmax1,z+nmax1
*enddo
*else
*do,j,0,ns-1,1,
t=1+num1+(diff*(p1-1))+(num2*j)
mat,np+j+1
e,t+botmax,1+num1+diff+(num2*j),1+num1+diff+(num2*j)+nmax1,t+botmax+nmax1
*enddo
*do,j,0,ns-1,1,
        t=1+num1+(num2*j)
        e,t+botmax,t+botmax+diff,t+botmax+diff+nmax1,t+botmax+nmax1,
        *if,p1,gt,1,then
                *get,emaxi,elem,,num,max
                egen,p1-1,diff,emaxi,
        *endif
*enddo
*endif
```

! Generates elements on the girder if a top partial panel exists on the deck
```
*if,tpwidth,eq,0,then
*do,j,0,ns-1,1
z=botmax-diff+1+num1+(num2*j)-diff
mat,np+j+1
e,z,z+diff,z+diff+nmax1,z+nmax1
*enddo
*else
*do,j,0,ns-1,1,
t=botmax-(diff*2)+1+num1+(num2*j)
mat,np+j+1
e,t,t+(diff*2)+topmax,t+(diff*2)+topmax+nmax1,t+nmax1
*enddo
*do,j,0,ns-1,1,
```

```
            t=botmax+1+num1+topmax+(num2*j)
            mat,np+j+1
            e,t,t+diff,t+diff+nmax1,t+nmax1,
            *if,c1,gt,1,then
                    *get,emaxi,elem,,num,max
                    egen,c1-1,diff,emaxi,
            *endif
*enddo
*endif
*get,emax2,elem,,num,max
egen,temp1,nmax1,emax1+1,emax2,1,
```

! **Model curbs**
! **cw = width of curbs**
! **cd = depth of curbs**
! **excurb = modulus of elasticity of curbs**
! **Curbs are modeled as beam elements running along the edges of the bridge between**
! **abutments. The curbs are connected to the deck with rigid constraint equations**

```
nsel,all
*ask,curbs,Curbs(1 for yes,0 for no)?,0
*if,curbs,eq,1,then
*ask,cw,Enter the curb width,8
*ask,cd,Enter the curb depth,21
*get,elmax,ELEM,,num,max
*get,nmax2,node,,num,max
*if,bpwidth,gt,0,then
nsel,S,node,,node(0,0,0)
nsel,A,node,,node(0,0,0)+diff-1
*if,p1,gt,1,then
*do,k,0,p1-1,1
nsel,A,node,,node(0,0,0)+(diff*k)
nsel,A,node,,node(0,0,0)+diff-1+(diff*k)
*enddo
*endif
*endif

*if,tpwidth,gt,0,then
*if,bpwidth,eq,0,then
nsel,S,node,,node(0,sb,0)
nsel,A,node,,node(0,sb,0)+diff-1
*else
nsel,A,node,,node(0,sb,0)
nsel,A,node,,node(0,sb,0)+diff-1
*endif
*if,c1,gt,1,then
```

```
*do,k,0,c1-1,1
nsel,A,node,,node(0,sb,0)-(diff*k)
nsel,A,node,,node(0,sb,0)+diff-1-(diff*k)
*enddo
*endif
*endif

*if,bpwidth,eq,0,then
*if,tpwidth,eq,0,then
nsel,S,node,,node(0,0,0)
nsel,A,node,,node(0,0,0)+diff-1
*else
nsel,A,node,,node(0,0,0)
nsel,A,node,,node(0,0,0)+diff-1
*endif
*endif

*if,tpwidth,eq,0,then
nsel,A,node,,botmax-diff+1
nsel,A,node,,botmax
*endif

*do,I,1,np,1,
*do,k,1,temp-1,1,
nsel,A,node,,((I-1)*(temp+1)*diff)+1+(diff*k)
nsel,A,node,,((I-1)*(temp+1)*diff)+diff+(diff*k)
*enddo
*enddo
ngen,2,nmax2,all,,,,,(pt/2)+5
et,3,beam4
r,3,cw*cd,(cd*cw**3)/12,(cd**3*cw)/12,cd,cw,
rmore, ,cw*cd**3*(1/3-0.21*(cd/cw)*(1-((cd**4)/(12*cw**4))))
real,3
*ask,excurb,Enter the modulus of elasticity of the curbs in psi,1800000
uimp,np+ns+1,EX,,,excurb
mat,np+ns+1
type,3
*do,I,1,np,1,
*do,k,1,temp-2,1,
e,((I-1)*(temp+1)*diff)+1+(diff*k)+nmax2.((I-1)*(temp+1)*diff)+1+(diff*k)+nmax2+diff
e,((I-1)*(temp+1)*diff)+diff+(diff*k)+nmax2.((I-
1)*(temp+1)*diff)+diff+(diff*k)+nmax2+diff
*enddo
*enddo
*do,I,1,np-1,1,
```

```
e,(I*diff*(temp+1))+nmax2-(diff*2)+1,(I*diff*(temp+1))+diff+nmax2+1
e,(I*diff*(temp+1))+nmax2-diff,(I*diff*(temp+1))+nmax2+(diff*2)
*enddo

*if,bpwidth,eq,0,then
e,1+nmax2,1+nmax2+diff
e,diff+nmax2,(diff*2)+nmax2
*else
*if,p1,gt,1,then
*do,I,1,p1-1,1
e,1+botmax+nmax2+(diff*(I-1)),1+botmax+nmax2+(diff*I)
e,diff+botmax+nmax2+(diff*(I-1)),diff+botmax+nmax2+(diff*I)
*enddo
e,1+botmax+nmax2+(diff*(p1-1)),1+nmax2+diff
e,diff+botmax+nmax2+(diff*(p1-1)),(diff*2)+nmax2
*else
e,1+botmax+nmax2,1+nmax2+diff
e,diff+botmax+nmax2,(diff*2)+nmax2
*endif
*endif

*if,tpwidth,eq,0,then
e,botmax-(diff*2)+1+nmax2,botmax-diff+1+nmax2
e,botmax-diff+nmax2,botmax+nmax2
*else
*if,c1,gt,1,then
*do,I,1,c1-1,1
e,botmax+1+topmax+nmax2+(diff*(I-1)),1+topmax+diff+botmax+nmax2+(diff*I)
e,diff+botmax+topmax+nmax2+(diff*(I-1)),(diff*2)+topmax+botmax+nmax2+(diff*I)
*enddo
e,botmax-(diff*2)+1+nmax2,botmax+1+topmax+nmax2
e,botmax-diff+nmax2,botmax+topmax+diff+nmax2
*endif
*endif

*do,I,1,np,1,
*do,k,1,temp-1,1,
zz=((I-1)*(temp+1)*diff)+(diff*k)
cerigid,zz+1,zz+1+nmax2
cerigid,zz+diff,zz+diff+nmax2
*enddo
*enddo

*if,bpwidth,gt,0,then
cerigid,1+botmax,1+botmax+nmax2
```

```
cerigid,diff+botmax,diff+botmax+nmax2
*if,p1,gt,1,then
*do,I,1,p1-1,1,
cerigid,1+botmax+(diff*I),1+botmax+(diff*I)+nmax2
cerigid,(diff*(I+1))+botmax,(diff*(I+1))+botmax+nmax2
*enddo
*endif
*else
cerigid,1,1+nmax2
cerigid,diff,diff+nmax2
*endif

*if,tpwidth,gt,0,then
cerigid,botmax+1+topmax,botmax+1+topmax+nmax2
cerigid,diff+botmax+topmax,botmax+diff+topmax+nmax2
*if,c1,gt,1,then
*do,I,1,c1-1,1,
cerigid,1+botmax+topmax+(diff*I),1+botmax+topmax+(diff*I)+nmax2
cerigid,(diff*(I+1))+botmax+topmax,(diff*(I+1))+botmax+nmax2+topmax
*enddo
*endif
*else
cerigid,botmax-diff+1,botmax-diff+1+nmax2
cerigid,botmax,botmax+nmax2
*endif

*endif


nsel,all
! Loads are modeled as concentrated loads.
! X(i) = X-coordinate of the ith load
! Y(i) = Y-coordinate of the ith load
! Q12(i) = Value of ith load
*dim,X,array,50
*dim,Y,array,50
*dim,Q12,array,50
        *ask,q,Enter the number of loads,1
        *do,i,1,q,1
                *ask,X(i),Enter X-coordinate of load %i%
                *ask,Y(i),Enter Y-coordinate of load %i%
                *ask,Q12(i),Enter the value of load %i%
        *enddo

! This segment calculates the element number upon which the concentrated load falls.
```

! The program then calculates the area of that element and applies the concentrated
! load as a pressure over the entire surface of the element. The value of the pressure
! load is the concentrated load divided by the element area

```
*dim,count,array,50
*dim,foz,array,50
*dim,mox,array,50
*dim,moy,array,50

*do,i,1,q,1
        Xo=X(i)
        Yo=Y(i)
        F=Q12(i)

*if, Xo,ge,0,then
*if,Xo,le,ld,then
xel=Xo*num1/ld
*elseif,Xo,le,(pl*cos(theta*pi/180))-rd,then
xel=num1+((Xo-ld)*num2*(ns-1)/((pl*cos(theta*pi/180))-ld-rd))
*elseif,Xo,le,(pl*cos(theta*pi/180))
xel=num1+(num2*(ns-1))+((Xo-(pl*cos(theta*pi/180))-rd)*num3/((pl*cos(theta*pi/180))-
rd))
*endif
*endif

*if,nint(xel),lt,xel,then
xel=nint(xel)+1
*else
xel=nint(xel)
*endif

*if,Yo,ge,(Xo*tan(theta*pi/180))+bpwidth,then
*if,Yo,lt,(Xo*tan(theta*pi/180))+sb-tpwidth,then
*do,k,1,np,1
*if,Yo,lt,(Xo*tan(theta*pi/180))+bpwidth+(k*pw),exit
*enddo
/com,'k' is the panel in which the load lies
yel1=(Yo-(Xo*tan(theta*pi/180))-bpwidth-((k-1)*pw))
yel=(yel1*temp/pw)+((k-1)*temp)
*elseif,Yo,le,(Xo*tan(theta*pi/180))+sb,then
yel1=Yo-(Xo*tan(theta*pi/180))-sb+tpwidth
*if,c1,gt,1,then
yel=yel1*c1/bpwidth
*else
yel=1
*endif
```

```
*endif
*elseif,Yo,ge,(Xo*tan(theta*pi/180)),then
yel1=Yo-(Xo*tan(theta*pi/180))
*if,p1,gt,1,then
yel=yel1*p1/bpwidth
*else
yel=1
*endif
*endif

*if,yel,gt,1,then
*if,nint(yel),lt,yel,then
yel=nint(yel)+1
*else
yel=nint(yel)
*endif
*endif
```

! Assigning element numbers to selected elements
```
*if,Yo,ge,(Xo*tan(theta*pi/180))+bpwidth,then
*if,Yo,lt,(Xo*tan(theta*pi/180))+sb-tpwidth,then
elemnum=xel+((yel-1)*(diff-1))
*elseif,Yo,le,(Xo*tan(theta*pi/180))+sb,then
elemnum=elmax2+xel+((yel-1)*(diff-1))
*endif
*elseif,Yo,ge,(Xo*tan(theta*pi/180)),then
elemnum=elmax1+xel+((yel-1)*(diff-1))
*endif

*if,elemnum,gt,1,then
arelem=(nx(nelem(elemnum,2))-nx(nelem(elemnum,1)))*(ny(nelem(elemnum,4))-
ny(nelem(elemnum,1)))
f1=F/arelem

*do,t,1,i,1
        *if,count(t),ne,elemnum,then
                        *if,count(t),eq,0,then
                                dot=0
                                *do,z,1,t,1
                                        *if,count(z),eq,elemnum,then
                                                dot=dot+1
                                        *endif
                                *enddo
                                *if,dot,eq,0,then
                                        count(t)=elemnum
```

```
                                 foz(t)=f1
                           *endif
                      *endif
                *else
                      foz(t)=foz(t)+f1
                *endif
*enddo
*endif
*enddo

num=0
*do,L,1,50,1
      *if,count(L),ne,0,then
            num=num+1
      *endif
*enddo

finish
```

! This segment applies the loads as pressures on surface of the elements
```
/solu
*do,t,1,num,1
      sfe,count(t),2,pres,,-foz(t)
*enddo
```

! Displacement constraints for the abutments. The bridge is assumed to be simply
! supported
```
*if,bpwidth,gt,0,then
*do,j,0,ns-1,1,
d,1+botmax+num1+(num2*j)+(nmax1*temp1),UX,,,,,UY,UZ
*enddo
*else
*do,j,0,ns-1,1,
d,1+num1+(num2*j)+(nmax1*temp1),UX,,,,,UY,UZ
*enddo
*endif
*if,tpwidth,gt,0,then
*do,j,0,ns-1,1,
d,nmax1-diff+1+num1+(num2*j)+(nmax1*temp1),UX,,,,,UZ
*enddo
*else
*do,j,0,ns-1,1,
d,botmax-diff+1+num1+(num2*j)+(nmax1*temp1),UX,,,,,UZ
*enddo
*endif
```

```
/psf,pres,norm,3
/pbf,defa,,1
/psymb,dot,1
/pbc,all,1
gplot
```

! If user is satisfied with model, enter solution=1
```
*ask,solution,Proceed with solution? (1 for yes/0 for no),1
*if,solution,eq,1,then
```
! Solve the system for the given loading
```
/solu
solve
finish
```

! Enter the postprocessor module. The output from the program is redirected to the file
! "Results.dat" which resides in the current home directory of the user. Girder nodes at
! midspan are selected and the midspan deflections and bending stresses at these nodes
! are printed in the output. In the case of skew bridges, the midspan is along the skew
! centerline. The girder nodes at midspan are located at the bottom of each girder.
```
/post1
!plnsol,U,Z,0,1
*dim,nodenum,array,ns
*dim,nodenumX,array,ns
*dim,nodenumY,array,ns
*dim,nodedisp,array,ns
*dim,stressY,array,ns
*dim,Label,char,1,4
/post1
nodenum(1)=node(ld,ld*tan(theta*pi/180)+(sb/2),-(pt/2)-bt)
nodenumX(1)=nx(nodenum(1))
nodenumY(1)=ny(nodenum(1))
nodedisp(1)=uz(nodenum(1))
*get,stressY(1),NODE,nodenum(1),S,Y
*do,j,1,ns-1,1,
nodenum(j+1)=node(ld,ld*tan(theta*pi/180)+(sb/2),-(pt/2)-bt)+(num2*j)
nodenumX(j+1)=nx(nodenum(j+1))
nodenumY(j+1)=ny(nodenum(j+1))
nodedisp(j+1)=uz(nodenum(j+1))
*get,stressY(j+1),NODE,nodenum(j+1),S,Y
*enddo
Label(1,1)='X-coord'
Label(1,2)='Y-coord'
Label(1,3)='UZ'
Label(1,4)='Ben-Stress'
/out,Results,dat,,
```

```
/com,Midspan Girder Deflections (in inches)
/com,------------------------------------
/com
/com
*vwrite,Label(1,1),Label(1,2),Label(1,3)
(A10,A10,A10)
*vwrite,nodenumX(1),nodenumY(1),nodedisp(1)
(F8.2,' ',F8.2,' ',F8.4)
/com,
/com,
/com,Midspan Girder Bending Stresses (in ksi)
/com,--------------------------------------
/com,(Negative stress denotes compression)
/com
*vwrite,Label(1,1),Label(1,2),Label(1,4)
(A10,A10,A10)
*vwrite,nodenumX(1),nodenumY(1),stressY(1)
(F8.2,' ',F8.2,' ',F8.4)
/out
*endif
```

APPENDIX C. DESCRIPTION OF THE USAGE OF THE PROGRAMS FOR ANALYZING GLULAM DECK BRIDGES AND GLULAM GIRDER BRIDGES

C.1 Procedure to use the programs with ANSYS

The programs for analyzing deck and girder bridges are called 'Deck.txt' and 'Girder.txt' respectively. These programs are text files and can be edited, if necessary, using any text editor like Wordpad, Notepad, or MS Word. To start using the program(s):

1. Copy the program 'Deck.txt' for analyzing longitudinal glulam deck bridges or 'Girder.txt' for analyzing glulam girder bridges, from the floppy disk to your hard drive.

2. Open ANSYS 5.5 on your computer. On the ANSYS utility menu bar, click on *File* and then on *Read input from*.

3. Click on the directory you have copied the program to and then click on the program name and press *OK*.

4. Enter the input data into the input windows that pop up on the screen. The model of the bridge is created, solved, and the output is redirected to the file 'Results.dat'. The output file can opened with any text editor like Wordpad, Notepad or MS Word. The results can also be exported into MS Excel to get a graphical representation of the results.

Note:

1. The X-axis is along the width of the bridge.

2. The Y-axis is in the longitudinal direction, which along the direction of traffic.

3. The Z-axis is positive upwards (according to the right-hand rule).

4. Input concentrated load acting in negative Z-direction as negative. Input pressure acting in negative Z-direction as positive. This is the same sign convention used in ANSYS.

C.2 Limitations of programs

1. These programs have to be used from within the ANSYS graphical user interface. The user has to start the ANSYS software first, then use the ANSYS menu option *Read input from*, and click on the required program file to start using the program.

2. The programs do not allow the user to go back and make changes to the input. If the user requires to change the input, the user should start the program again and enter the new input.

3. The output files only give a text representation of the desired results. To obtain results apart from those displayed in the output file, the user must have a moderate understanding of ANSYS. Click on *General Postprocessor* in the main menu, and then *Plot Results* to obtain contour plots of stresses and deflections.

C.3 User manual for preprocessor for right angled longitudinal glulam deck bridges

1. Enter number of deck panels = np

2. Enter panel width in inches = pw

3. Enter panel length in inches = pl

4. Enter panel thickness in inches = pt

5. Enter number of stiffeners = *ns*

6. Enter beam width in inches = *bw*

7. Enter beam thickness in inches = *bt*

8. Enter modulus of elasticity of stiffener beams in psi = *exb*

9. Enter shear modulus of stiffener beams in psi = *sb*
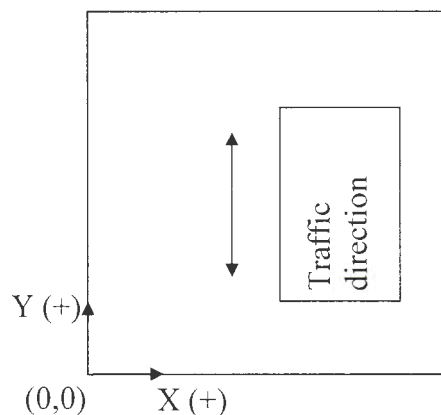
Fig. C.1 Sketch showing the Cartesian coordinate axes for the longitudinal glulam deck bridge.

10. Do all panels have same material properties (1 for yes/0 for no)? = *ans*

11. If *ans*=1, then

    Enter longitudinal E of panels in psi = *eyp*

    Enter transverse E of panels in psi = *exp*

    Enter shear modulus of panels in psi = *gp*

12. If *ans*=0, then

    Enter longitudinal E of panel *i* in psi = *eypanel(i)*

Enter transverse E of panel *i* in psi = *expanel(i)*

Enter shear modulus of panel *i* in psi = *gpanel(i)*

where *i* = panel number.

13. Enter curbs (1 for yes, 0 for no)? = *curbs*

14. If *curbs*=1, then

Enter the curb width in inches = *cw*

Enter the curb depth in inches = *cd*

15. Enter patch loads (0) or concentrated loads (1)? = *loads*

16. If *loads*=1, then

Enter the number of loads = *q*

Enter X-coordinate of load *i* = *X(i)*

Enter Y-coordinate of load *i* = *Y(i)*

Enter the value of load *i* = *Q12(i)*

where *i* = load number.

17. If *loads*=0, then

Enter number of patch loads = *q1*

Enter lower left X-coordinate of patch load *i* = *X1(2*i-1)*

Enter lower left Y-coordinate of patch load *i* = *Y1(2*i-1)*

Enter upper right X-coordinate of patch load *i* = *X1(2*i)*

Enter upper right Y-coordinate of patch load $i = X1(2*i)$

Enter pressure value of load $i = Q11(i)$

where i = load number.

18. Proceed with solution? (1 for yes/0 for no) = *solution*

19. If *solution*=1, proceed from 20.

20. Enter panel no. for stress results (0 to stop) = *panelnum(T)*

21. If the user has indicated the panel number for which he desires the stress results, then

Do you want the complete listing? (1 for yes/0 for no) = *outp*

If *outp*=1, the complete listing is stored in the file *Results.dat*, else only the

maximum values of stresses for the desired panels are stored in the file.

22. At this point, the post processor is executed and the output is routed to the file

Results.dat; the output consists of midspan deflections of the panel edges, panel

stresses for user-inputed panel numbers, and midspan stiffener beam bending stresses.

23. The output file *Results.dat* can be opened using any text editor like MS Word,

Wordpad or Notepad. To obtain a graphical representation of the results in MS Excel,

select the results by pressing down the left button on the mouse (or holding down the

SHIFT key on the keyboard) and releasing when selection is over. Right click on the

mouse and choose 'Copy'. Open MS Excel, right click on any cell, and select 'Paste'.

Click on 'Data' option on the main menu bar, then on 'Text to columns'. Click

'Finish' and now, the data is available for graphing on Excel. .

C.4 User manual for preprocessor for glulam girder bridges

1. Enter angle of skew in degrees (-ve if left end ahead) = *theta*

2. Enter span of bridge in inches = *sb*

3. Enter full panel width in inches = *pw*

4. Enter panel length in inches along *theta* = *pl*

5. Enter panel thickness in inches = *pt*

6. Enter number of girders = *ns*

7. Enter girder width in inches = *bw*

8. Enter girder thickness in inches = *bt*

9. Enter left overhang in inches = *ld*

10. Enter right overhang in inches = *rd*

11. Partial panels between abutments? (0 for no, 1 for yes) = *ppanel*

12. If *ppanel*=1, then

    Enter width of partial panel at the bottom = *bpwidth*

    Enter width of partial panel at the top = *tpwidth*

13. Enter element size in inches = *elsize*

14. Do all panels have same material properties (1 for yes/0 for no)? = *ans*

15. If *ans*=1, then

Enter longitudinal E of panels in psi = *exp*

Enter transverse E of panels in psi = *eyp*

Enter shear modulus of panels in psi = *gp*
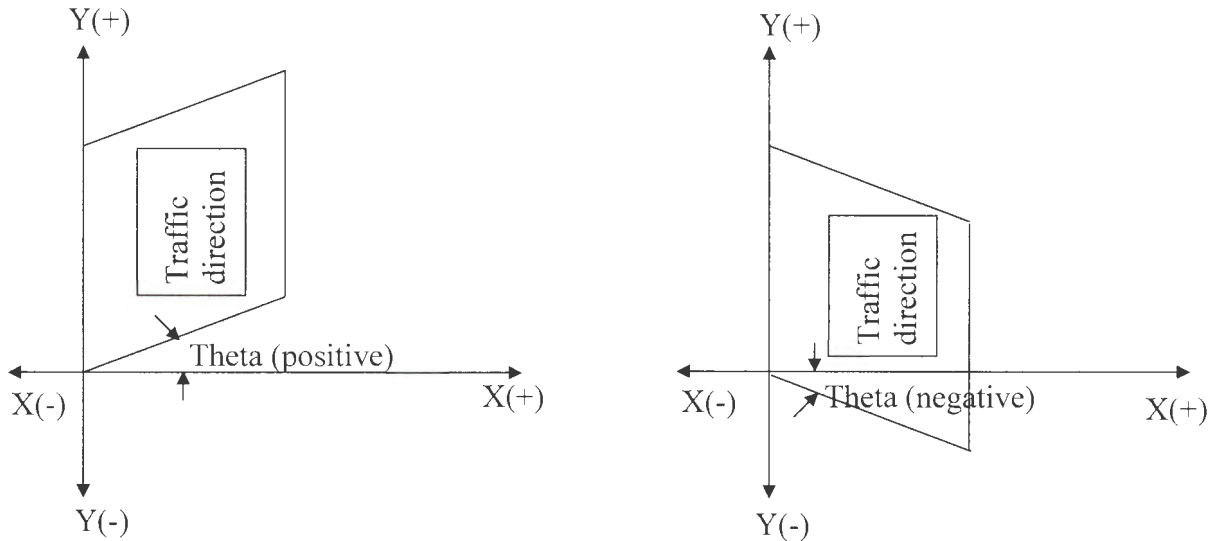
Fig. C.2 Sketch showing the Cartesian coordinate axes for the glulam girder bridge.

16. If *ans*=0, then

    Enter longitudinal E of full panel *i* in psi = *expanel(i)*

    Enter transverse E of full panel *i* in psi = *eypanel(i)*

    Enter shear modulus of full panel *i* in psi = *gpanel(i)*

where *i* = full panel number. Numbering starts from bottom.

17. Do all stringers have same material properties? (1 for yes,0 for no) = *ans*

18. If *ans*=1, then

Enter longitudinal modulus of elasticity of girders in psi = *exb*

Enter transverse modulus of elasticity of girders in psi = *eyb*

Enter shear modulus of girders in psi = *gb*

19. If *ans*=0, then

Enter longitudinal E of girder *jt* in psi = *eystring(jt)*

Enter transverse E of girder *jt* in psi = *exstring(jt)*

Enter shear modulus of girder *jt* in psi = *gstring(jt)*

where *jt* = girder number. Numbering of girders start from left.

20. Enter curbs (1 for yes,0 for no)? = *curbs*

21. If *curbs*=1, then

Enter the curb width in inches = *cw*

Enter the curb depth in inches = *cd*

Enter the modulus of elasticity of the curbs in psi = *excurb*

22. Enter the number of loads = *q*

23. Enter X-coordinate of load *i* = *X(i)*

Enter Y-coordinate of load *i* = *Y(i)*

Enter the value of load *i* = *Q12(i)*

where *i* = load number (i.e. from 1 to *q*)

24. Proceed with solution? (1 for yes/0 for no) = *solution*

25. If *solution*=1, proceed from 26.

26. At this point, the postprocessor is executed and the output is routed to the file *Results.dat*; the output consists of midspan deflections and maximum bending stresses of the girders measured at nodes located at the bottom of the girders.

27. These output results can be imported into an Excel spreadsheet and graphs can be plotted.

C.5 Example problem for longitudinal timber glulam deck bridge

To illustrate the use of the preprocessor, the Angelica Bridge has been chosen with the loading being Load Case 1. The material properties and load positions on the bridge have already been described in Section 3.2.

Input data:

Enter number of deck panels = 8

Enter panel width in inches = 53.4

Enter panel length in inches = 356

Enter panel thickness in inches = 14.4

Enter number of stiffeners = 4

Enter beam width in inches = 6.75"

Enter beam thickness in inches = 4.25"

Enter modulus of elasticity of stiffener beams in psi = 1625000

Enter shear modulus of stiffener beams in psi = 80000

Do all panels have same material properties? (1 for yes/0 for no) = 1

Enter longitudinal E of panels in psi = 1700000

Enter transverse E of panels in psi = 240000

Enter shear modulus of panels in psi = 100000

Enter curbs (1 for yes/0 for no)? = 1

Enter the curb width in inches = 8

Enter the curb depth in inches = 21

Enter patch loads (0) or concentrated loads (1)? = 1

Enter the number of loads = 4

Enter X-coordinate of load 1 in inches = 237.6

Enter Y-coordinate of load 1 in inches = 150.7

Enter the value of load 1 in pounds = -13225

Enter X-coordinate of load 2 in inches = 237.6

Enter Y-coordinate of load 2 in inches = 205.3

Enter the value of load 1 in pounds = -13225

Enter X-coordinate of load 3 in inches = 309.6

Enter Y-coordinate of load 3 in inches = 150.7

Enter the value of load 1 in pounds = -13225

Enter X-coordinate of load 4 in inches = 309.6

Enter Y-coordinate of load 4 in inches = 205.3

Enter the value of load 1 in pounds = -13225

Proceed with solution? (1 for yes/0 for no) = 1

Enter panel no. for stress results = 6

Enter panel no. for stress results = 0

Do you want the complete listing? (1 for yes/0 for no) = 0

Output data:

The output is redirected to the file *Results.dat*. The output for the above input data looks as below. This output can be copied and pasted onto a spreadsheet. This data, however gets pasted into one cell of the sheet. To change the format, i.e., to have individual cells containing individual data, the user should do the following.

1. Click on *'Data'* on the menu of the Excel sheet.

2. Click on *'Text to columns'*.

3. In the window that pops up, click on *'Fixed width'*. Click on *'Next'* and then *'Finish'*.

Note:

1. Negative value of deflection implies downward deflection, i.e. deflection in the negative Z- direction.

2. Stress results are the bottom stresses for the shell elements and the beam elements.

3. Negative stress indicates compression and positive stress indicates tension.

Table C.1 Midspan panel deflections (inches).

X-coord	Y-coord	UZ
0.0	178	-0.0050
53.4	178	-0.0219
53.4	178	-0.0229
106.8	178	-0.0740
106.8	178	-0.0762
160.2	178	-0.1783
160.2	178	-0.1835
213.6	178	-0.3699
213.6	178	-0.3844
267.0	178	-0.4686
267.0	178	-0.4654
320.4	178	-0.4198
320.4	178	-0.4001
373.8	178	-0.1382
373.8	178	-0.1258
427.2	178	-0.0357

where   X-coord = x-coordinate in inches,
            Y-coord = y-coordinate in inches, and
            UZ = deflection in inches.

Panel – 6

Maximum longitudinal stress in panel is 541.5 psi.
 At location
           X = 317.40 in.
           Y = 186.90 in.

Maximum tranverse stress in panel is 369.2 psi.
 At location
           X = 304.05 in.
           Y = 151.30 in.

Maximum shear stress in panel is 230.3 psi.
 At location
           X = 304.05 in.
           Y =   26.70 in.

Table C.2 Stresses in stiffener beam close to midspan (psi).

X-coord	Y-coord	Bend. St
26.7	213.6	-6.2314
53.4	213.6	-165.6233
80.1	213.6	-6.3236
106.8	213.6	-251.8111
133.5	213.6	-0.9444
160.2	213.6	-432.4457
186.9	213.6	-3.4234
213.6	213.6	446.6235
240.3	213.6	585.4596
267.0	213.6	591.5234
293.7	213.6	469.9873
320.4	213.6	990.9346
347.1	213.6	0.6345
373.8	213.6	-767.6345
400.5	213.6	-31.2345

where X-coord = x-coordinate in inches,

Y-coord = y-coordinate in inches, and

Bend. St = bending stress in psi (bottom fiber of beam).

C.6 Example problem for glulam girder bridge

To illustrate the use of the preprocessor, the Cow Gulch Bridge has been chosen with the loading being Load Case 1. The material properties and load positions on the bridge have already been described in Section 5.2.

Input data:

Enter angle of skew in degrees (-ve if left end ahead) = 0

Enter span of bridge in inches = 468

Enter full panel width in inches = 48

Enter panel length in inches along theta = 336

Enter panel thickness in inches = 5.125

Enter number of girders = 6

Enter girder width in inches = 8.75

Enter girder thickness in inches = 28.5

Enter left overhang in inches = 23.75

Enter right overhang in inches = 23.625

Partial panels near abutments? (0 for no/ 1 for yes) = 1

Enter width of partial panel at the bottom = 36

Enter width of partial panel at the top = 0

Enter element size in inches = 12

Do all panels have same material properties (1 for yes/0 for no)? = 1

Enter longitudinal E of panels in psi = 1800000

Enter transverse E of panels in psi = 130000

Enter shear modulus of panels in psi = 100000

Do all stringers have same material properties? (1 for yes/0 for no) = 1

Enter longitudinal modulus of elasticity of girders in psi = 2000000

Enter transverse modulus of elasticity of girders in psi = 240000

Enter shear modulus of elasticity of girders in psi = 106000

Enter curbs (1 for yes. 0 for no)? = 1

Enter the curb width in inches = 8

Enter the curb depth in inches = 8

Enter the modulus of elasticity of the curbs in psi = 1800000

Enter the number of loads = 6

Enter X-coordinate of load 1 in inches = 192

Enter Y-coordinate of load 1 in inches = 14.6

Enter the value of load 1 in pounds = -7050

Enter X-coordinate of load 2 in inches = 264

Enter Y-coordinate of load 2 in inches = 14.6

Enter the value of load 2 in pounds = -7050

Enter X-coordinate of load 3 in inches = 192

Enter Y-coordinate of load 3 in inches = 203.6

Enter the value of load 3 in pounds = -9205

Enter X-coordinate of load 4 in inches = 264

Enter Y-coordinate of load 4 in inches = 203.6

Enter the value of load 4 in pounds = -9205

Enter X-coordinate of load 5 in inches = 192

Enter Y-coordinate of load 5 in inches = 258.6

Enter the value of load 5 in pounds = -9205

Enter X-coordinate of load 1 in inches = 264

Enter Y-coordinate of load 1 in inches = 258.6

Enter the value of load 1 in pounds = -9205

Proceed with solution? (1 for yes/0 for no) = 1

Output data:

The output is redirected to the file *Results.dat*. The output for the above input data looks as below. This output can be copied and pasted onto a spreadsheet. This data, however gets pasted into one cell of the sheet. To change the format, i.e., to have individual cells containing individual data, the user should do the following.

4. Click on *'Data'* on the menu of the Excel sheet.

5. Click on *'Text to columns'*.

6. In the window that pops up, click on *'Fixed width'*. Click on *'Next'* and then *'Finish'*.

Note:

4. Negative value of deflection implies downward deflection, i.e. deflection in the negative Z- direction.

5. Stress results are the bottom stresses for the shell elements.

6. Negative stress indicates compression and positive stress indicates tension.

Table C.3 Midspan girder deflections (inches).

X-coord	Y-coord	UZ
23.75	240	-0.0276
81.47	240	-0.1497
139.2	240	-0.3194
196.93	240	-0.4846
254.65	240	-0.4794
312.38	240	-0.2925

where   X-coord = x-coordinate in inches,

Y-coord = y-coordinate in inches, and

UZ = deflection in inches.

Table C.4 Midspan girder bending stresses (psi).

X-coord	Y-coord	Ben-Stre
23.75	240	60.4
81.47	240	219.5
139.2	240	490.9
196.93	240	739.6
254.65	240	721.6
312.38	240	360.5

where   X-coord = x-coordinate in inches,

Y-coord = y-coordinate in inches, and

Ben-Stre = bending stress in psi (bottom fiber of beam).

REFERENCES

1. Ritter, Michael A. 1990. *Timber Bridges: Design, Construction, Inspection, and Maintenance*. Forest Service – USDA. Washington, DC.

2. Funke, Robert W. 1986. *Behavior of Longitudinal Glued Laminated Timber Deck Bridges*. MS Thesis submitted to Iowa State University. Ames, IA.

3. Bisat, Aida T. 1996. *Analytical Modeling of Longitudinal Glued Laminated Timber Deck Bridges*. MS Creative Component submitted to Iowa State University. Ames, IA.

4. Hajdu, T. 1994. *Dynamic Behavior of Longitudinal Glued Laminated Timber Deck Bridges*. MS Thesis submitted to Iowa State University. Ames, IA.

5. Bhari, S. 1997. *Static Behavior of Glued Laminated Timber Stringer Bridges*. MS Creative Component submitted to Iowa State University. Ames, IA.

6. Dlabola, J. 1995. *Dynamic Behavior of Glued Laminated Timber Stringer Bridges*. MS Thesis submitted to Iowa State University. Ames, IA.

7. Johnson, M.R. 1993. *Dynamic Behavior of Stress Laminated Timber Bridges*. MS Thesis submitted to Iowa State University. Ames, IA.

8. Swanson Analysis Systems, Inc. 1992. *ANSYS User's Manual for Revision 5.0 – Procedures*. Houston, PA.

9. American Institute of Timber Construction. 1987. *Design Standard Specifications for Structural Glued Laminated Timber of Softwood Species*. AITC 117-87-Design. Englewood, CO.

10. Sanders, W.W., Jr.; Klaiber, F.W.; Wipf, T.J. 1985. *Load Distribution in Glued Laminated Longitudinal Timber Deck Highway Bridges*. Iowa State University – Engineering Research Institute. Ames, IA.

11. Swanson Analysis Systems, Inc. 1992. *ANSYS User's Manual for Revision 5.0 – Elements*. Houston, PA.

12. Desai, C.S.; Abel, J.F. 1972. *Introduction to Finite Element Method: A Numerical Method to Engineering Analysis*. Van Nostrand Reinhold, Inc. NY.

13. Jain, S.H. 2000. *Analytical Modeling and Preprocessor Development of Stress-laminated Timber Deck Bridges.* MS Thesis submitted to Iowa State University. Ames, IA.

14. American Institute of Timber Construction. 1985. *Timber Construction Manual*. 3d ed. John Wiley and Sons, Inc. NY.

15. Szilard, R. 1973. *Theory and Analysis of Plates – Classical and Numerical Methods*. Prentice-Hall, Inc. Englewood Cliffs, NJ.

ACKNOWLEDGEMENTS