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Examination of Response of a Skewed Steel Bridge Superstructure During Deck Placement

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The response of a 74.45-m (244-ft 0-in.) skewed bridge to the placement of the concrete deck was monitored to compare measured and predicted behavior. This comparison was completed to (*a*) determine theoretical deflections and rotations with analytical models for comparison to actual deformations monitored during construction; (*b*) compare the results of various levels of analysis to determine the adequacy of the methods; and (*c*) examine variations on the concrete placement sequence to determine the most efficient deck placement methods. Two levels of analysis were used to achieve the objectives. Level 1 was a two-dimensional finite element grillage model analyzed with STAAD/Pro. Level 2 was a three-dimensional finite element model analyzed with SAP2000. These studies are discussed and findings are presented.

The design of skewed bridges is becoming more commonplace in the United States. Site constraints in urban areas dictate the use of more extreme abutment and pier orientations. In addition, skewed bridges are common at highway interchanges, river crossings, and other extreme grade changes where skewed geometries are necessary because of limitations in space. Research into the behavior of skewed bridges has been limited. Studies utilizing field testing generally focused on determining distribution factors and the influence of the angle of skew on the behavior of the deck (*1, 2*). Several studies have been conducted utilizing laboratory testing as a means to validate an analytical model, and they included sensitivity studies to predict the effects of specific parameters on behavior (*3–9*). However, no field studies to date have examined the response of skewed bridges during construction.

Bridges with small skew angles typically are designed as modified right-angle structures. The girders in a right-angle structure are placed perpendicular to the abutment. While it is efficient to model bridges with an angle of skew less than 20° as right-angle structures, torsional moments and rotations, shears, and support reactions caused by more severe angles of skew cannot be efficiently portrayed (*7, 10*). During construction of bridges without skewed supports, the screed and concrete are aligned and placed perpendicular to the centerline of the superstructure. This allows for an even distribution of the wet concrete dead load to the supporting girders (Figure 1*a*). Screed position is more important during placement of the concrete deck on a skewed bridge. The alignment of the screed can affect the final geometry of the structure. Concrete placed perpendicular to the centerline of the bridge will result in an uneven distribution of dead loads across the superstructure. Because the abutments are skewed, the weight of the wet concrete placed by the screed near the acute corner will cause girders near this corner to deflect more than girders near the obtuse corner (L1 > L2 in Figure 1*b*). Differential deflections that result under this dead load cause gross rotation of the bridge cross section. To attempt to compensate for this rotation and the problems that may result once the concrete has hardened, the girders can be erected out-of-plumb. AASHTO and the National Steel Bridge Alliance have developed a method for erecting girders in skewed bridges that theoretically accounts for these rotations (*11*); however, there is no known research that has evaluated the effectiveness of this method. The method consists of originally erecting the girders with the webs plumb as indicated in Figure 2*a*. The top portion of the cross frame is then connected to the top of the webs of its adjacent girders (G1 and G2 in Figure 2*a*). The bottom of G2 is deformed until its transverse stiffener connection holes are in line with the cross-frame connection holes (Figure 2*b*). The bottom of G1 is then deformed until its stiffener connection holes line up with the bottom cross-frame connection holes (Figure 2*b*). Next, a cross frame is positioned between G2 and G3, and the bottom of G3 is deformed until its stiffener connection holes line up with the bottom of the cross frame. This process continues until all cross frames are in place and all girders are rotated an amount equal and opposite the anticipated rotation due to the deck weight (*11*).

While differential deflections and the subsequent rotations imposed on skewed bridges during deck construction have not significantly affected their performance in the past, more efficient design practices and the use of high-strength steels have increased the necessity for this research (*11*). To properly design a skewed structure, its behavior during construction must be better understood. Specifically, this paper discusses the influence of the 12-h concrete deck placement process to determine deck placement sequencing effects on the deflected shape and stress levels in a skewed steel bridge superstructure that was unshored during the pour. Theoretical deflections and rotations were determined with analytical models for comparison with actual deformations monitored during construction. Several variations on the concrete placement sequence were examined to determine their effects on response.

STRUCTURE DESCRIPTION

Pennsylvania Department of Transportation Structure 28 is a single-span, composite, steel, plate girder bridge located on an extension to Interstate 99 in central Pennsylvania. The bridge is 74.45 m (244 ft 3 in.) long with a skew of about 55° as indicated in Figure 3*a*. Each girder is constructed with $17.5 \times 2{,}400$ mm ($^{11}\!/_{16} \times 94$ in.) web plates and flange plates that range between 50.8×609.6 mm (2×24 in.) and 76.2×762 mm (3×30 in.). The girders are braced with X-shaped cross frames that consist of either (4) L6x6x$^5\!/_8$ or (3) L6x6x$^5\!/_8$

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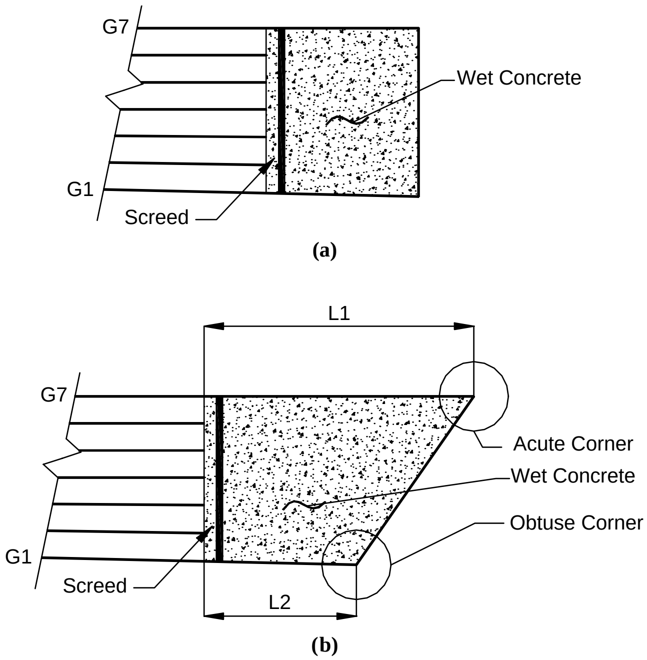

FIGURE 1 Screed position on (*a*) right-angle structure, (*b*) skewed structure.

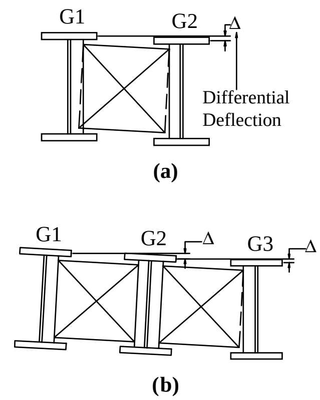

FIGURE 2 Erection of rotated girders (*1*): (*a*) girders erected in vertical position and cross frame erected with rotation; (*b*) bottom of G2, then G1, deformed until stiffener holes line up with cross-frame holes.

and a W8x24 bottom chord. End cross frames are K frames with a W16x45 top chord, (2) L6x6x⅝ and a W8x24 bottom chord. Cross frames are staggered at each abutment. Girders G1, G2, G6, and G7 are supported by nonguided expansion bearings at the abutments, while the remaining girders rest on guided expansion bearings at the west abutment and fixed bearings at the east abutment.

Girders were erected with web plates out-of-plumb at the abutments and at midspan. The out-of-plumb angle ranged between 0.57° and 0.61° with a corresponding lateral displacement of about 25.4 mm (1 in.) at the top and bottom flanges (Figure 3*b*). The girders were fabricated in the plumb position; however, the cross frames were fabricated to force the webs out-of-plumb by an amount equal and opposite the anticipated rotation due to the deck weight.

Concrete deck placement began at the east abutment and proceeded perpendicular to the centerline of the bridge with two screeds (Figure 4). The screeds, each spanning half the width of the bridge, were staggered 7.62 m (25 ft) apart in an attempt to place the wet concrete parallel to the skewed abutments so that the differential deflections between adjacent girders would be minimized. Screed rails were attached to G1, G4, and G7.

FIELD TESTING

The structure was monitored during the entire 12-h deck placement process. Longitudinal strains and girder displacements were measured with strain transducers manufactured by Bridge Diagnostics, Inc. (BDI), and linear variable differential transformers (LVDTs). Instruments were placed on the structure as indicated in Figure 5*a*, with the BDI transducers measuring stresses in the girder flanges and in individual cross-frame members and the LVDTs measuring lateral displacements of the girder webs at the abutments. In addition to data supplied by the strain transducers and LVDTs, global geometric data were also collected from traditional surveys before and after the deck placement process (Figure 5*b*). The Pennsylvania Department of Transportation also completed surveys with a three-dimensional Cyrax laser scanner system. This system has a reported accuracy of less than 6 mm (0.2 in.) (*12*) and would perform one scan of the structure surface in 10 min. In addition to the surface scans, laser targets were attached to the girder bottom flanges at five locations to track deformations during the deck pour (Figure 5*b*).

NUMERICAL ANALYSIS

Two models were developed to determine the accuracy of numerical methods for predicting the response of skewed bridges during construction. The first model was a two-dimensional grillage model devel-

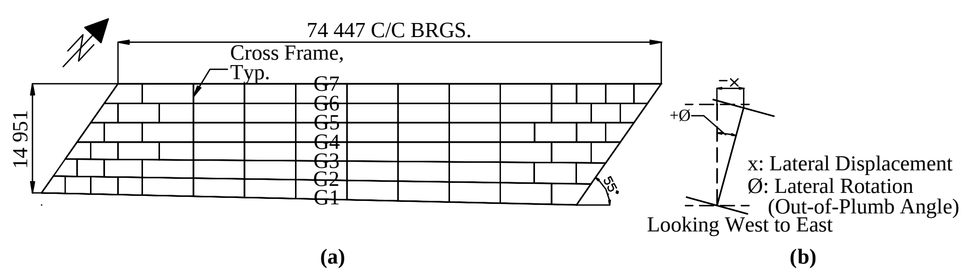

FIGURE 3 (*a*) Plan view (in millimeters); (*b*) initial lateral girder deformations. (C/C BRGS. = centerline to centerline of the bearings.)

FIGURE 4 Twelve-hour concrete deck pour.

oped in STAAD/Pro. The second model was a three-dimensional finite element model developed in SAP2000.

The three-dimensional structure was reduced to a two-dimensional grillage model and analyzed with STAAD/Pro (*13*). Section properties for each plate girder were calculated, and the girders were modeled as noncomposite frame elements (Table 1). The interior and end cross frames, made up of diagonal members and top and bottom chords, were condensed into frame elements that incorporated section properties from all the members. Moments of inertia for these elements were determined with only the top and bottom chords of the cross frames; however, all members were used for calculating the cross-sectional area (*13*). All girders at the east abutment and G1, G2, G6, and G7 at the west abutment were supported by pins that restrained vertical, longitudinal, and lateral translation. G3, G4, and G5 at the west abutment were restrained against vertical and lateral translation only. Girder rotations could not be explicitly modeled in the two-dimensional model. Loads used in this model mimicked those applied to the actual structure during construction. Wet concrete was modeled as uniform loads acting along each girder. Progression of the concrete pour and movement of the screed across the deck was modeled in four load stages with point loads for the screed on G1, G4, and G7 and moments that accounted for its overhang on G1 and G7. The first load stage represented the self-weight of the steel. The second load stage included the screed and deck wet concrete load on a quarter span of the bridge. The next stage included these loads placed onto half the bridge. The fourth stage included these loads placed on three-fourths of the bridge, while the final stage included loads from the entire wet concrete deck after the screed exited the bridge. Deflections, rotations, and strains determined from this analysis were compared with field data and with the three-dimensional model.

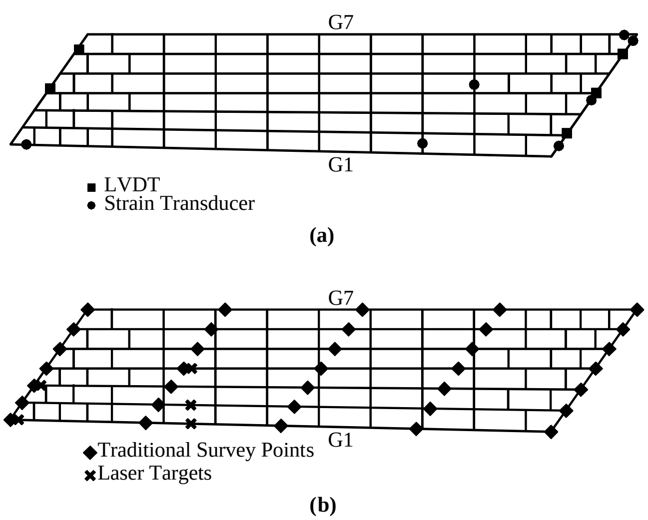

FIGURE 5 Instrument plan: (*a*) strain transducer/LVDT locations, (*b*) survey locations.

The three-dimensional finite element model was constructed and analyzed with SAP2000. Nodes were placed at the top and bottom of the girder web and at the neutral axis. Shell elements were used to model the girder webs while space frame elements were used to model girder flanges, stiffeners, and the cross frames. Boundary conditions used for the three-dimensional model were identical to those used in the grillage model and these restraints were placed at the bottom node of each girder web. The coordinates of the rotated girders were calculated to explicitly model the initial rotations forced into the girders to compensate for anticipated rotation due to the deck pour (Figure 3*b*). The cross frames were rigidly connected to the girders. Lateral displacements and corresponding girder rotations at the abutments before placement of the deck are presented in Table 2. To attempt to provide an accurate distribution of wet concrete loads to the girders, wet concrete was modeled as shell elements connected to the girder flanges with rigid links. Shell elements require a modulus of elasticity; however, wet concrete has virtually no stiffness. To model the concrete as accurately as possible, a modulus of elasticity of 68.9 MPa (10 kips/in.^{2}) was assigned to the shell elements. Deflections, rotations, and strains were compared with predictions from the grillage model and with field results.

DECK POUR SEQUENCING STUDIES

Information from past projects in Pennsylvania has indicated that deck pour sequencing can have a significant impact on the final deflected shape of a structure. Factors that influence the final deflected shape include positioning of the screed and the sequence in which the wet concrete was placed. To examine the level of influence of deck pour sequencing on skewed bridge response, the three-dimensional SAP2000 model was modified to examine the effect of placing the concrete both parallel to the abutments and perpendicular to the bridge centerline.

The screed was placed perpendicular to the centerline of the bridge for Case A and parallel to the abutments for Case B. Case A was used during the actual placement process.

Loads were applied to the three-dimensional model in the same manner in which they were applied to the grillage model. Again, four stages, each representing about one-fourth of the complete deck pour, were used for the wet concrete loads.

DISCUSSION OF RESULTS

Displacement data measured by the LVDTs are presented and compared with predicted values from the SAP2000 model before and after placement of the concrete deck. Displacements from the numerical models are compared with measured values obtained during the placement process. A preliminary comparison of the cross-frame

TABLE 1 Section Properties

GIRDER		1-6			7		
SECTION PROPERTIES		Thickness (mm)	Width (mm)	Neutral Axis[1] (mm)	Thickness (mm)	Width (mm)	Neutral Axis (mm)
1	TOP FLANGE	38.10	609.60	1725.15	38.10	609.60	1725.15
1	BOTTOM FLANGE	50.80	609.60		50.80	609.60	
1	WEB	17.46	2387.60		17.46	2387.60	
2	TOP FLANGE	57.15	609.60	1774.49	63.50	609.60	1728.44
2	BOTTOM FLANGE	76.20	609.60		76.20	609.60	
2	WEB	17.46	2387.60		17.46	2387.60	
3	TOP FLANGE	63.50	609.60	1793.91	63.50	609.60	1793.91
3	BOTTOM FLANGE	76.20	685.80		76.20	685.80	
3	WEB	17.46	2387.60		17.46	2387.60	
4	TOP FLANGE	57.15	609.60	1774.49	69.85	609.60	1811.58
4	BOTTOM FLANGE	76.20	609.60		76.20	762.00	
4	WEB	17.46	2387.60		17.46	2387.60	
5	TOP FLANGE	38.10	609.60	1725.15	63.50	609.60	1793.91
5	BOTTOM FLANGE	50.80	609.60		76.20	685.80	
5	WEB	17.46	2387.60		17.46	2387.60	
6	TOP FLANGE	N/A[2]	N/A	N/A	63.50	609.60	1728.44
6	BOTTOM FLANGE	N/A	N/A		76.20	609.60	
6	WEB	N/A	N/A		17.46	2387.60	
7	TOP FLANGE	N/A	N/A	N/A	38.10	609.60	1725.15
7	BOTTOM FLANGE	N/A	N/A		50.80	609.60	
7	WEB	N/A	N/A		17.46	2387.60	

[1]Neutral axis measured from top flange.
[2]G1-G6 contained five different sections, G7 contained seven different sections

TABLE 2 Lateral Deflections and Rotations Before Deck Placement

Girder	Abutment Location	Top of Web* (mm)	Bottom of Web* (mm)	Neutral Axis* (mm)	Lateral Rotation* (°)
G1	East	-26.11	-1.68	-12.48	0.59
G1	West	23.47	-1.30	9.66	-0.59
G2	East	-25.60	-1.25	-12.02	0.59
G2	West	24.03	-0.74	10.21	-0.59
G3	East	-25.17	-0.97	-11.67	0.58
G3	West	24.89	0.00	11.01	-0.60
G4	East	-24.82	0.00	-10.97	0.60
G4	West	25.25	0.00	11.16	-0.61
G5	East	-24.41	1.22	-10.11	0.62
G5	West	25.60	0.00	11.32	-0.61
G6	East	-24.23	1.65	-9.79	0.62
G6	West	26.42	0.51	11.96	-0.62
G7	East	-24.28	2.54	-9.32	0.64
G7	West	27.03	0.76	12.38	-0.63

*See Figure 3 for sign convention

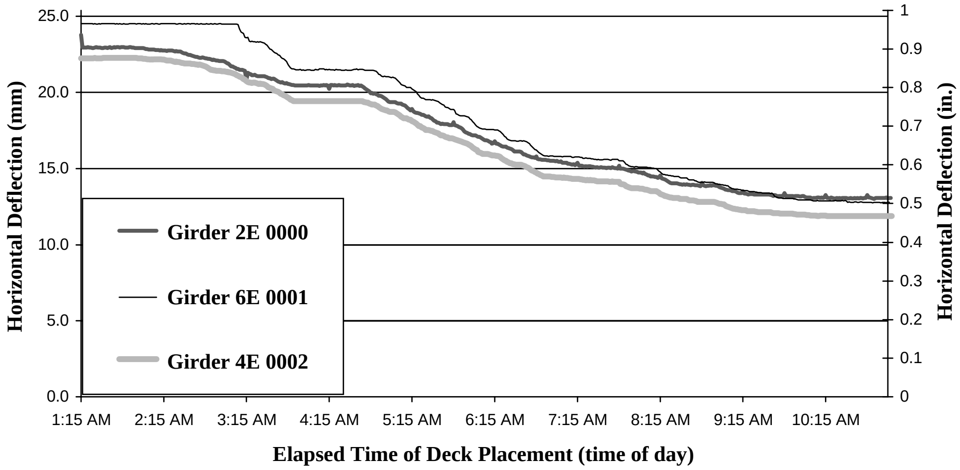

FIGURE 6 Measured horizontal deflections.

forces, support reactions, and vertical displacements between Cases A and B is also presented. Strain data from the BDI transducers are not presented here.

Measured Lateral and Vertical Displacements

Preliminary field results indicated that the girders were not plumb after deck placement. Figure 6 presents horizontal deflections of G2, G4, and G6 at the east abutment throughout the deck pour. The girders were expected to rotate about 25.4 mm (1 in.); however, the average rotation was only 12.7 mm (0.5 in.). While lateral displacements did not match what was predicted, measured vertical displacements were in good agreement with design predictions (Figure 7).

Numerical Model Verification

Vertical deflections for G1 at the completion of the pour are presented in Figure 8. As previously stated, actual boundary conditions for all girders at the east abutment and for G1, G2, G6, and G7 at the west abutment were modeled as pinned supports that restrained vertical, longitudinal, and lateral translations. G3, G4, and G5 at the west abutment were guided expansion bearing that restrained vertical and lateral translations. These modifications were made to the numerical models during the calibration process to minimize differences between predicted and measured results.

It was observed that vertical deflections determined from the three-dimensional finite element model were 0.3% to 10% higher than measured deflections. The grillage model was not as accurate, overpredicting vertical deflections by 25% to 38%. Lateral displacements determined from the three-dimensional model were generally nonconservative; however, the average ratio of predicted to measured lateral displacements was 1.2 (Table 3).

Preliminary Cross-Frame Member Force Comparison

Forces in top and bottom horizontal cross-frame members for Cases A and B are compared in Figures 9 through 13. Preliminary analyses indicate there is a slight difference in the maximum forces in the horizontal cross-frame members during Stage 2 through Stage 5 of construction.

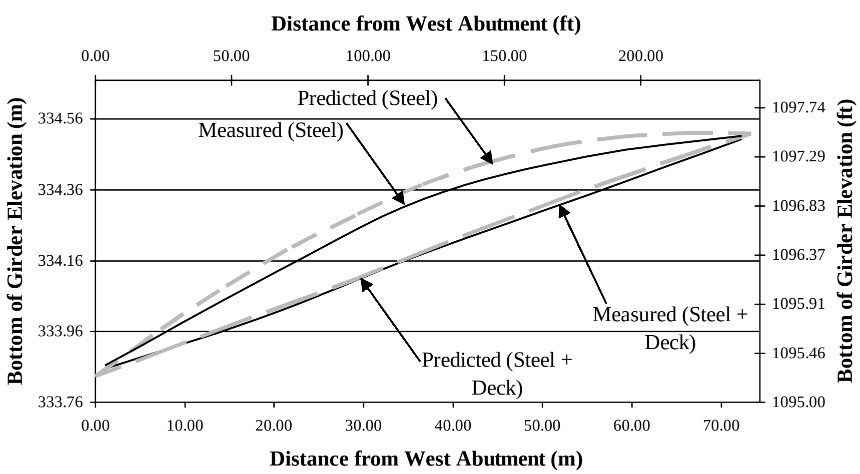

FIGURE 7 Bottom of girder elevations, G1.

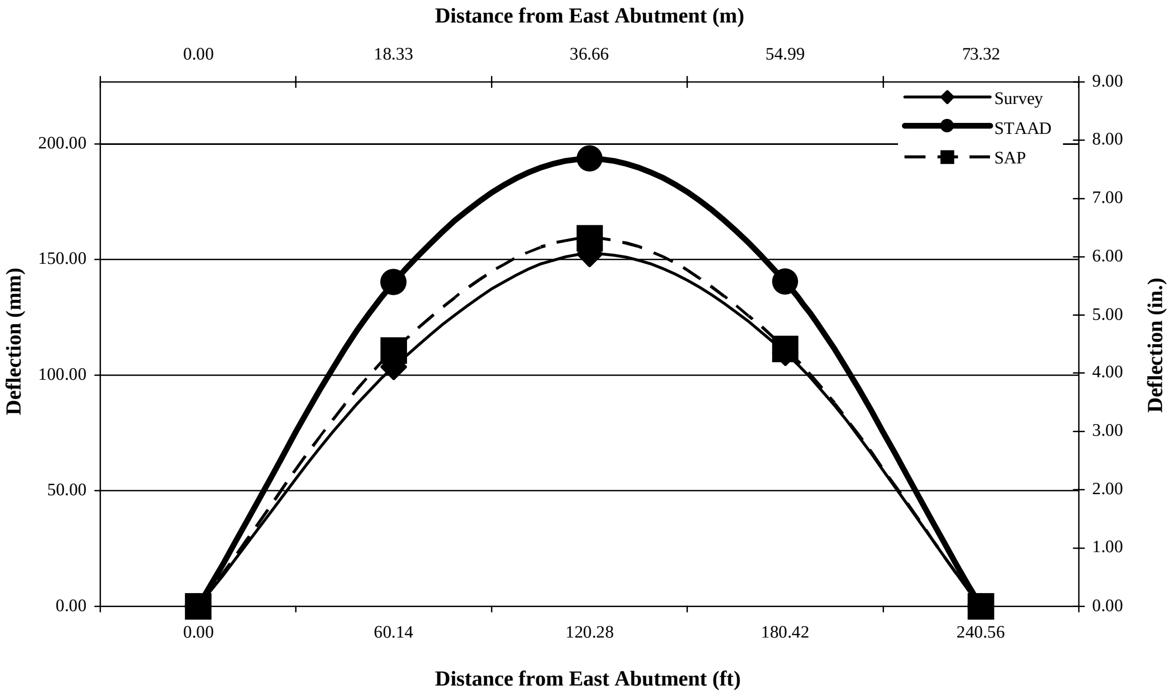

FIGURE 8 Wet concrete deflection at completion of pour, G1.

Preliminary Comparison of Support Reactions and Maximum Vertical Displacements

Variations in support reactions are presented in Table 4 for intermediate stages of construction. It was observed numerically that support reactions for Case A were 0.2% to 26% higher than the reactions for Case B during Stages 2 through 4. It was also observed numerically that maximum vertical displacements obtained from Case A were 1.51% to 7.30% higher than Case B during intermediate stages of construction (Table 5).

CONCLUSIONS

A theoretical and experimental study was performed to determine the effect of varying deck placement on skewed superstructures. The work is in progress and further results will be presented in the near future. The following preliminary conclusions are valid for single-span, simply supported, composite steel-concrete skewed bridges: (*a*) the three-dimensional model yielded more accurate results than the two-dimensional grillage model; (*b*) varying deck placement has no significant impact on the forces in the cross frames if the concrete is assumed to remain plastic; (*c*) placing the deck perpendicular to the centerline of the bridge leads to higher support reactions during the intermediate stages of construction; and (*d*) displacements at intermediate stages of the pour also tend to be higher when the deck is placed perpendicular to the centerline of the bridge.

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TABLE 3 Comparison Between Measured and Predicted Lateral Displacements

Girder	Abutment	Predicted - 3-D model		Measured		Predicted/Measured	Design	
		(in.)	(mm)	(in.)	(mm)		(in.)	(mm)
G2	East	0.61	15.5	0.51	13.0	1.20	0.87	22.1
G4	East	0.62	15.7	0.47	11.9	1.32	0.92	23.4
G4	West	-0.47	11.9	-0.44	11.2	1.06	0.83	21.1
G6	East	0.66	16.8	0.50	12.7	1.32	0.96	24.4

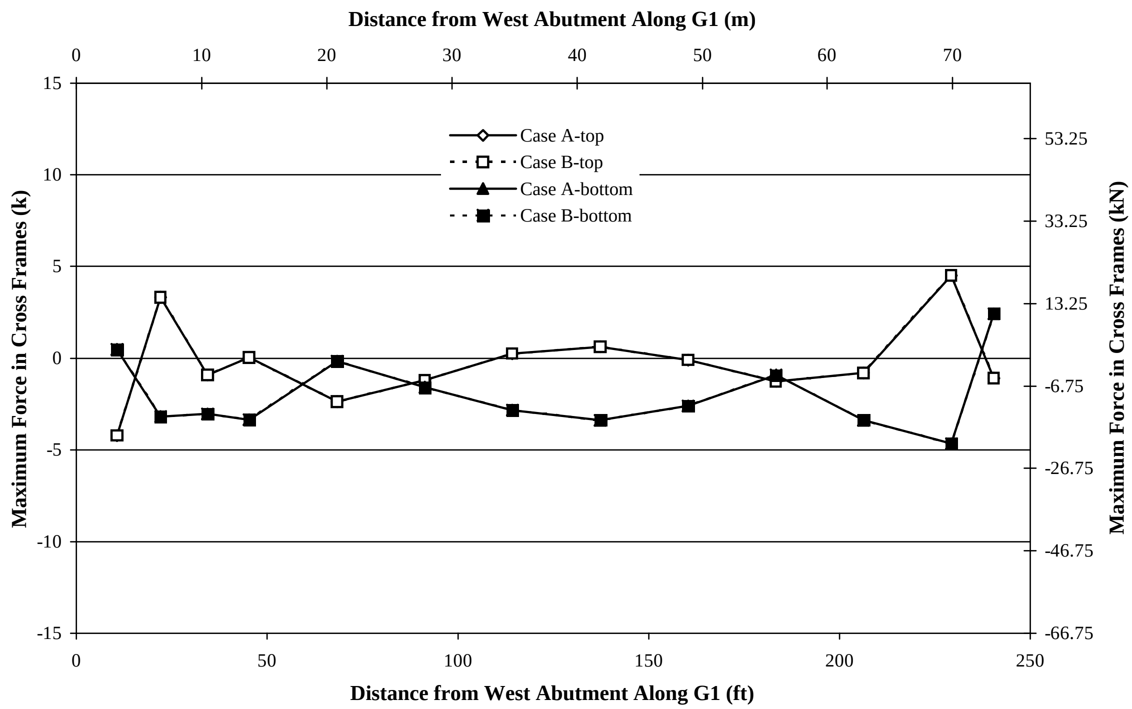

FIGURE 9 Maximum forces in cross frames: Stage 1.

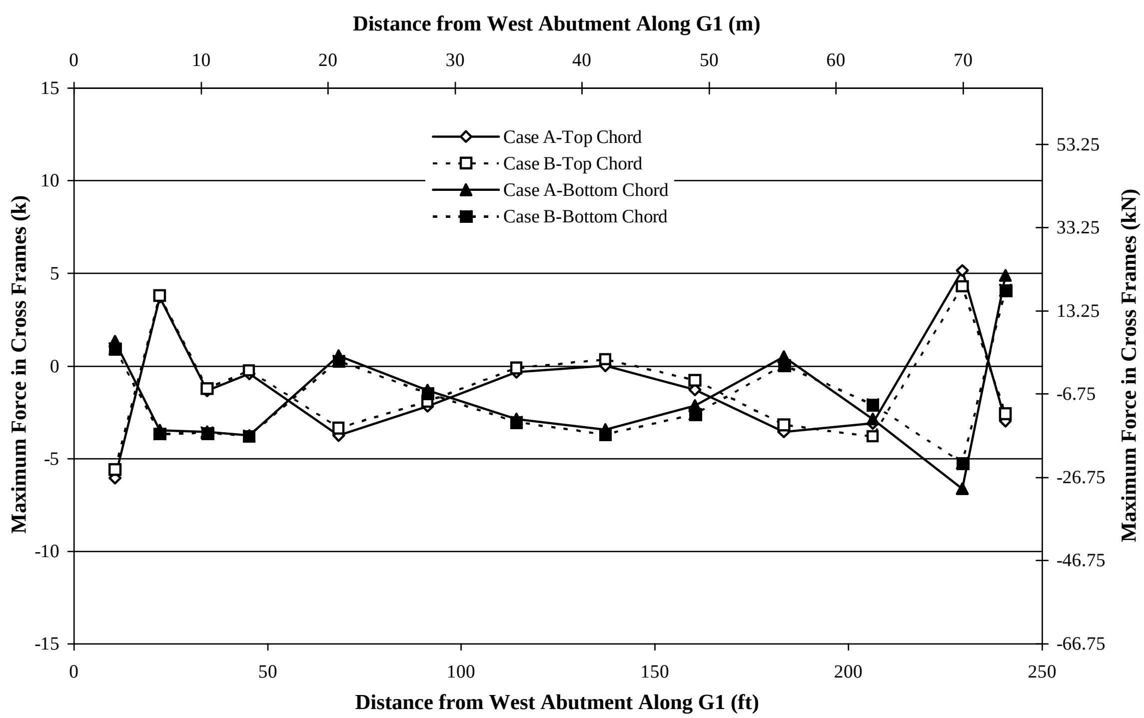

FIGURE 10 Maximum forces in cross frames: Stage 2.

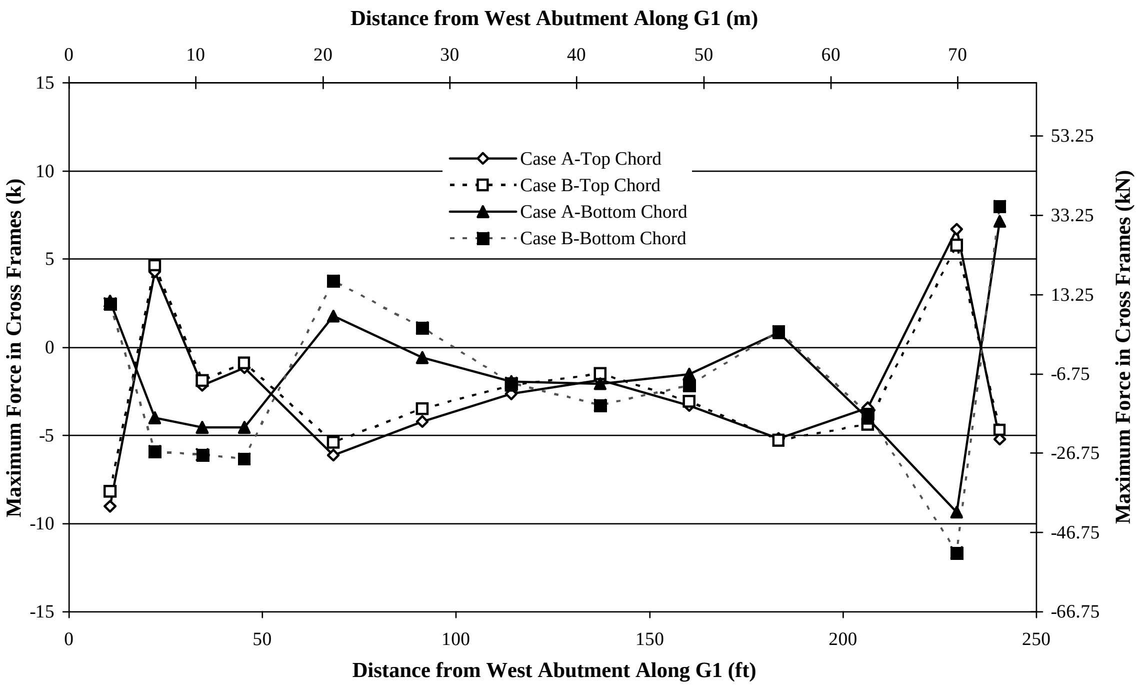

FIGURE 11 Maximum forces in cross frames: Stage 3.

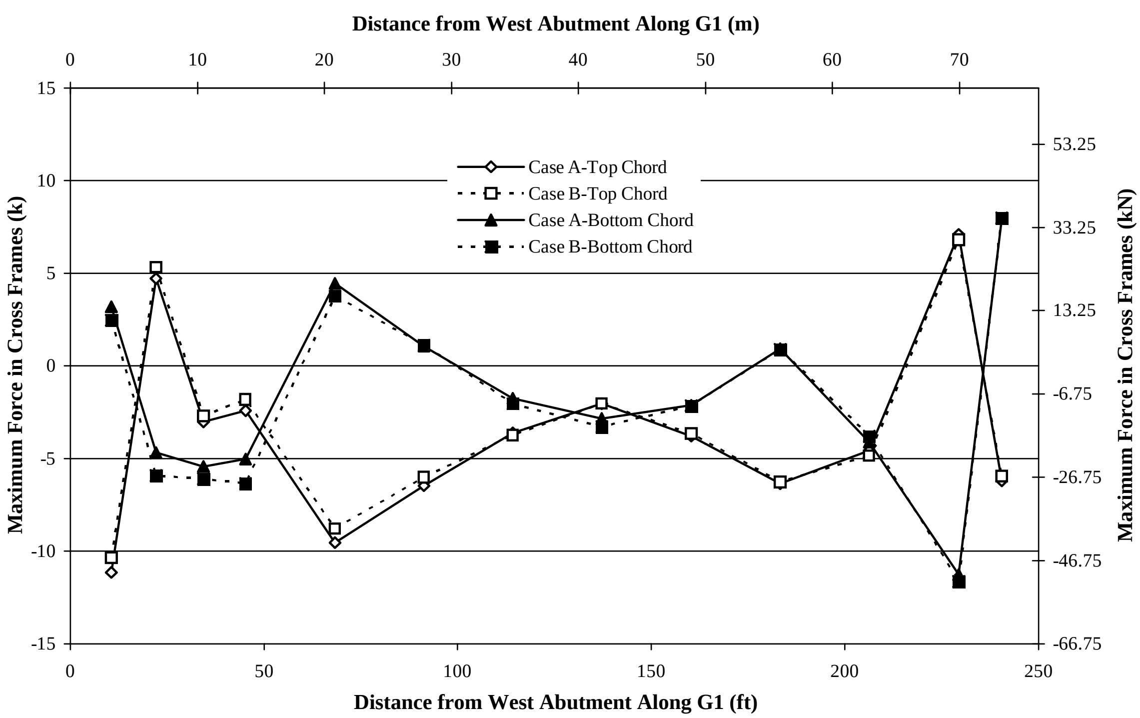

FIGURE 12 Maximum forces in cross frames: Stage 4.

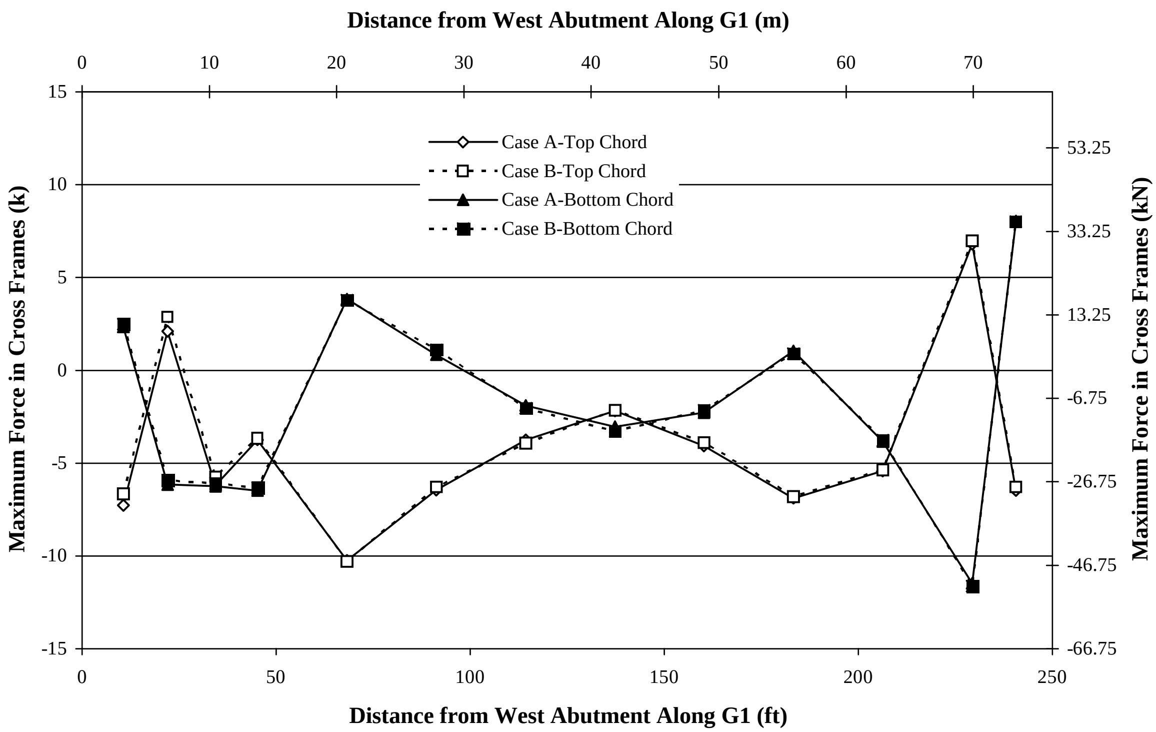

FIGURE 13 Maximum forces in cross frames: Stage 5.

TABLE 4 Comparison of Abutment Support Reactions

Girder	Abutment	Ratio of Support Reactions (Case A/Case B) Stage 2	Stage 3	Stage 4
1	West	1.03	1.04	1.03
2	West	1.00	1.01	1.02
3	West	1.55	1.16	1.00
4	West	0.99	1.01	1.04
5	West	1.02	1.03	1.04
6	West	1.04	1.05	1.05
7	West	1.07	1.13	1.13
1	East	0.99	0.99	1.00
2	East	1.03	1.02	1.01
3	East	1.04	1.03	1.02
4	East	1.06	1.03	1.01
5	East	1.07	1.04	1.01
6	East	1.06	1.03	1.01
7	East	1.06	1.03	1.01

TABLE 5 Maximum Vertical Displacements

	Displacements (mm)					
	Stage 2		Stage 3		Stage 4	
Girder	Case A	Case B	Case A	Case B	Case A	Case B
G1	-138.4	-136.4	-196.1	-192.0	-253.2	-249.4
G2	-138.9	-135.6	-198.4	-192.3	-255.0	-250.2
G3	-140.7	-135.6	-201.9	-193.8	-258.3	-252.7
G4	-143.3	-136.9	-206.8	-196.6	-263.4	-256.8
G5	-147.6	-139.4	-213.6	-201.4	-270.5	-262.9
G6	-152.7	-142.7	-221.7	-207.3	-279.4	-270.8
G7	-158.8	-147.3	-231.4	-214.6	-289.8	-280.4

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