Design and Field Monitoring of Horizontally Curved Steel Plate Girder Bridge

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Bridge 207 is a two-span horizontally curved steel plate girder bridge near Port Matilda, Pennsylvania. Although the curvature is not severe, the curvature combined with the unequal span balance caused an unusual distribution of force effects in the girders. A global twisting of the superstructure was caused by the unequal vertical deflections in the two spans. The computer program BSDI-3D was used to analyze the curved superstructure. To account for the out-of-plumb condition of the girders in their final condition, additional lateral flange bending moments were calculated. The magnitude of the additional lateral moments was a function of the vertical bending moments and the degree of twist in the girder. Field monitoring of the structure is focusing on the effects of curvature during construction. This is being accomplished by developing a detailed time line of superstructure erection and deck placement and through monitoring of the bridge by using vibrating wire strain gauges and tiltmeters positioned at critical locations on the girders and cross-frames. Field data were recorded before and after critical construction events, such as girder erection, cross-frame and formwork placement, and the deck pour. This information is being used to determine the effects of curvature on the cross-frames during construction by using axial stresses and strains and on the girders by using warping stresses and strains.

This paper presents issues raised during the design and construction phases of a two-span curved steel plate girder bridge. The bridge was designed in 2000 and was scheduled for completion in 2004. The design used commercially available finite element software and did not contain unusual details, materials, or degree of curvature. However, the combination of girder curvature with unequal span lengths did create a distribution of forces through the girder system that was not initially intuitively obvious. The resulting girder deflections and moments in this curved steel bridge are reminders that engineering intuition gained by experience in straight girder design is not always completely applicable to curved-girder design.

This paper also describes a method for incorporating the second-order lateral moment effects in the steel girders caused by the out-of-plumb position of the girders. Current software and analysis techniques do not directly address the lateral component of the vertical bending moment in an out-of-plumb girder. To include this second-order effect, a simple manual calculation method is described.

Field monitoring of the structure focused primarily on construction, including superstructure erection, placement of formwork, and reinforced concrete deck placement. To most accurately assess the effects of curvature on the structure, vibrating wire strain gauges and tiltmeters were used to measure strains and rotations at critical locations on the girders and cross-frames. A preliminary three-dimensional finite element model was used to determine instrument placement and anticipated values, and a detailed time line of the superstructure erection process and deck placement procedure was documented. Although the investigation is not complete, the following objectives will be reached at the end of this study. The scope of this study is limited to the one bridge being tested, and the objectives of the field study are as follows:

- To evaluate the effects of curvature on warping stresses and vertical and radial deformations during all phases of construction,
- To determine if a grillage model can accurately predict the effect of curvature on vertical bending and warping stresses and deformations during construction of the study bridge, and
- To explore the basis for the limits set forth in AASHTO Table 4.6.1.2.1-1 of the bridge design specifications (*1*) and determine if these limits are appropriate for the study bridge.

Some preliminary data from the field study are presented.

PROJECT DESCRIPTION

The SR 0220, Section C11 project includes the design of a 2.6-mi (4-km) section of PA-0220 (also known as I-99) on new alignment in Centre County, Pennsylvania. This new alignment will be a four-lane divided limited-access Interstate highway, which will form a northwestern loop around the borough of Port Matilda. The estimated construction cost was \$75 million, and construction was scheduled to be completed in 2004. The project includes 16 new bridges, of which Bridge 207 is the only steel plate girder structure.

Bridge 207 is a two-span continuous curved steel girder bridge. The bridge will carry two lanes of traffic from eastbound US-322 to northbound I-99. The bridge is shown in Figure 1, and Figure 2 is the bridge framing plan. Bridge details are as follows:

Span arrangement: Span 1 = 209 ft 8 in. (63.9 m); Span 2 = 271 ft 1 in. (82.62 m);
Bridge cross section: five steel plate girders spaced at 10 ft 8 in. (3.25 m);
Radius of curvature: approximately 1,921 ft (585.48 m) to the center girder;
Girder depth: 9 ft (2.74 m);
Girder material: ASTM A709 Grade 50 steel;
Deck: 9-in. (229-mm) concrete composite deck; and
Cross-frames: K-frames of WT sections spaced at approximately 18 ft (5.5 m).

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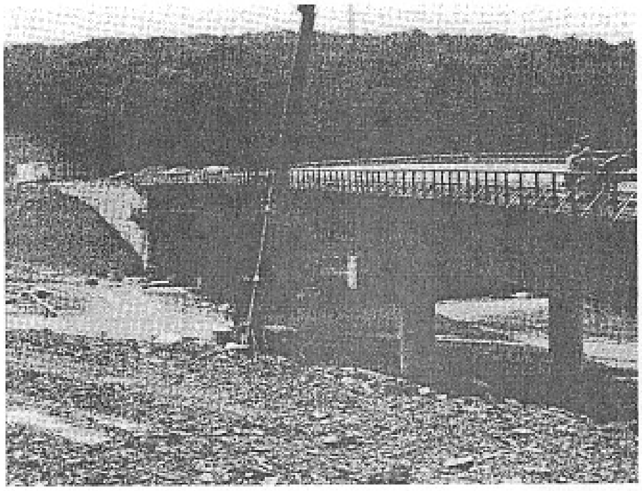

FIGURE 1 Elevation of Bridge 207 during construction.

A reinforced concrete two-column bent and reinforced concrete stub abutments on piles support the superstructure. Pot bearings were used at each support location.

BRIDGE ANALYSIS AND DESIGN

Understanding of the general behavior of steel girders curved in plan is crucial to the evaluation of the Bridge 207 analysis results.

Curved-Girder Behavior

Because the center of gravity of a single curved girder lies outside the line drawn between its supports, a curved girder supporting its own self-weight has a tendency to rotate about its longitudinal axis and roll toward its outside (longer) edge. This lack of stability of a single curved girder must be addressed during erection. A system of curved girders, connected by cross-frames along their length, can be self-supporting and stable if properly designed. The tendency of the curved girders to roll is resisted by the cross-frame forces within a girder system. Therefore, the cross-frames in a curved-girder bridge are considered to be main load-carrying members, critical to the overall stability of the structure. The forces in the cross-frames impart lateral bending moments into the girders, increasing the tip stresses in the flanges.

Method of Analysis

The finite element program BSDI-3D (Bridge Software Development International, Ltd., Coopersburg, Pa.) was used to analyze and design the Bridge 207 superstructure (*2*). BSDI-3D is a commercially available bridge analysis and design software package. The BSDI-3D model incorporates the depth of the girders and the actual cross-frame geometry, providing a complete three-dimensional model of the bridge superstructure. Iterating through analysis and design refined the girder plate sizes to create an optimal design. The shears, moments, and deflections in the girders were output for each iteration. The girders were designed by BSDI-3D by using the AASHTO load factor design method (*2*).

Analysis Results

It is prudent to study the results of each iteration of analysis to gain a sense for the correctness of the output. An engineer with structural modeling experience and good engineering judgment can review the relative magnitudes of forces and displacements in the girders to determine if the computer model is correctly quantifying the forces and displacements throughout the actual structure. When reviewing analysis output for curved-girder structures, an understanding of the unique behavior of these types of structures is essential. When the first iteration of analysis for Bridge 207 was complete, some of the resulting output exhibited unexpected trends. However, the unique behavior of curved-girder bridges explains the output trends and highlights the differences between these structures and tangent ones.

Girder Deflections

Table 1 is the steel dead load camber table from the final Bridge 207 model. Because the girders are curved in plan and the supports are aligned radially to the girders, each girder has a slightly different span length and deflection. The longer Span 2 exhibits the pattern of relative deflections that would be expected in most bridge superstructures—a direct positive relationship between span length and deflections. The longest girder, G1, shows the largest dead load deflection at midspan of Span 2, 4.87 in. (124 mm). The shortest girder, G5, shows the smallest dead load deflection at midspan of Span 2, 2.93 in. (74 mm).

However, the dead load deflections in the shorter span, Span 1, reveal the exact opposite trend. The longest girder, G1, actually had a peak negative (upward) deflection in this span equal to −0.96 in. (−24 mm). Conversely, the shortest girder in Span 1, G5, had the largest downward Span 1 dead load deflection, equal to 0.55 in. (14 mm). This counterintuitive result is caused by the unequal two-span curved-girder layout. Because Span 2 is significantly larger than Span 1, its deflection behavior dominates the entire superstructure system. The rolling effect in Span 2 causes the opposite effect in Span 1, effectively twisting the entire superstructure. The exaggerated downward deflections in G1, Span 2, reduce the downward deflections in G1, Span 1.

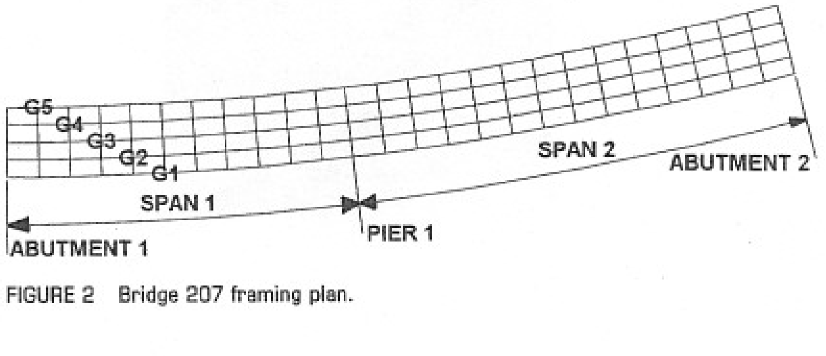

FIGURE 2 Bridge 207 framing plan.

TABLE 1 Steel Dead Load Cambers

			Span 1 = 214′ 6″ (65.38m)*								
Girder	Girder Length	Camber Units	1.10	1.20	1.30	1.40	1.50	1.60	1.70	1.80	1.90
G5	475′ 5″	English (in.)	0.27	0.46	0.55	0.52	0.38	0.18	−0.02	−0.15	−0.15
	(144.91m)	Metric (mm)	7	12	14	13	10	5	−1	−4	−4
G4	478′ 1″	English (in.)	0.20	0.33	0.35	0.28	0.12	−0.08	−0.25	−0.32	−0.24
	(145.72m)	Metric (mm)	5	8	9	7	3	−2	−6	−8	−6
G3	480′ 9″	English (in.)	0.13	0.19	0.15	0.03	−0.15	−0.35	−0.48	−0.49	−0.34
	(146.53m)	Metric (mm)	3	5	4	1	−4	−9	−12	−12	−9
G2	483′ 5″	English (in.)	0.05	0.04	−0.05	−0.22	−0.43	−0.62	−0.72	−0.66	−0.42
	(147.35m)	Metric (mm)	1	1	−1	−6	−11	−16	−18	−17	−11
G1	486′ 1″	English (in.)	−0.02	−0.10	−0.26	−0.47	−0.71	−0.91	−0.96	−0.84	−0.51
	(148.16m)	Metric (mm)	−1	−3	−7	−12	−18	−23	−24	−21	−13
			Span 2 = 266′ 3″ (81.15m)*								
Girder	Girder Length	Camber Units	2.10	2.20	2.30	2.40	2.50	2.60	2.70	2.80	2.90
G5	475′ 5″	English (in.)	0.49	1.19	1.95	2.54	2.89	2.93	2.62	2.00	1.11
	(144.91m)	Metric (mm)	12	30	50	65	73	74	67	51	28
G4	478′ 1″	English (in.)	0.62	1.46	2.34	3.01	3.39	3.41	3.05	2.32	1.28
	(145.72m)	Metric (mm)	16	37	59	76	86	87	77	59	33
G3	480′ 9″	English (in.)	0.76	1.73	2.73	3.47	3.88	3.89	3.47	2.63	1.45
	(146.53m)	Metric (mm)	19	44	69	88	99	99	88	67	37
G2	483′ 5″	English (in.)	0.90	2.01	3.12	3.93	4.37	4.36	3.88	2.94	1.62
	(147.35m)	Metric (mm)	23	51	79	100	111	111	99	75	41
G1	486′ 1″	English (in.)	1.03	2.29	3.52	4.40	4.87	4.85	4.30	3.26	1.80
	(148.16m)	Metric (mm)	26	58	89	112	124	123	109	83	46

*Span length measured along Girder G3.

Similarly, the smaller downward deflections in G5, Span 2, reduce the downward deflections in G5, Span 1, by a lesser amount.

Once understood, this behavior can be visualized as a global torsional effect on the steel girder superstructure. As the larger curved span rotates outward toward the longer girder, the shorter span attempts to rotate in the opposite direction. The resulting deflections in the shorter span display an inverse relationship between span length and dead load deflection. When this behavior was observed in the initial Bridge 207 BSDI-3D output, it became obvious that the girder reactions and moments would also vary from the typical results seen in tangent girders and symmetric curved-girder span arrangements. These specific results are described in the following.

Girder Reactions

The girder end reactions at the abutments display a trend similar to that seen in the deflections. Table 2 lists the reactions at each abutment. The abutment reactions in Span 2, the longer span, are positively related to the girder span lengths. The longest girder in Span 2, G1, has a self-weight dead load reaction of 105 kips (467 kN) at the abutment. The shortest girder in Span 2, G5, has a corresponding reaction of only 50 kips (222 kN). This pattern is in agreement with the intuitive notion that reactions should increase with increasing girder lengths. In addition, the large disparity in G1 and G5 reactions highlights the rolling behavior of the curved-girder system toward the outside girders.

However, the abutment reactions in Span 1, the shorter of the two spans, display the opposite trend. At this location, the G1 self-weight reaction is 23 kips (102 kN), and the shorter G5 reaction is 27 kips (120 kN). In the shorter curved span, Span 1, the girder reactions are inversely related to the span lengths of each girder. The tendency of the larger span to roll outward exaggerated the Span 2 reactions of the longer girders and reduced the Span 2 reactions of the shorter girders. Conversely, this created the opposite effect on the Span 1 reactions. As was seen in the girder deflections, the global twisting of the curved-girder system generated a pattern of reactions at Abutment 1 that differed from the expected trends.

Relative Moment Magnitudes

The positive moment distribution in the two spans displayed trends similar to the end reactions. The magnitudes of the Span 2 positive moments were directly related to the individual girder span lengths.

TABLE 2 Girder Dead Load Reactions at Abutments

Girder	Girder Length	Span 1 214′ 6″ (65.38m)*	Span 2 266′ 3″ (81.15m)*
G5	475′ 5″ (144.91m)	27 kips (120 kN)	50 kips (222 kN)
G4	478′ 1″ (145.72m)	26 kips (116 kN)	57 kips (254 kN)
G3	480′ 9″ (146.53m)	26 kips (116 kN)	61 kips (271 kN)
G2	483′ 5″ (147.35m)	23 kips (102 kN)	76 kips (338 kN)
G1	486′ 1″ (148.16m)	23 kips (102 kN)	105 kips (467 kN)

*Span length measured along Girder G3.

The peak Span 2 steel self-weight positive moment was 17,600 kip-ft (23,800 kN-m) in G1 and 9,000 kip-ft (12,200 kN-m) in G5. However, in the shorter Span 1, the peak steel self-weight positive moments were 3,800 kip-ft (5,100 kN-m) in both the innermost and outermost girders. Again, the torsional twisting of the steel girder system was responsible for these Span 1 positive moment patterns.

Locations of Points of Inflection

The twisting of the superstructure that has been revealed by the deflections, reactions, and moments also changes the locations of the points of inflection. In a tangent girder system or a curved-girder system with equal spans, the transverse lines connecting the points of inflection within a span are radial to the girders (or nearly so) in plan. However, the torsional behavior of Bridge 207 under steel self-weight skews the points of inflection from this radial geometry. The G1 point of inflection is near 0.55L of Span 1. The G5 point of inflection is at 0.63L of Span 1. This result has implications for setting the locations of bolted field splices and deck pour limits.

GIRDER DESIGN

Lateral Bending Effects in Curved Girders

Most steel plate girder bridges are constructed with steel cross-frames connecting the girders. These cross-frames help maintain the stability of the girders during erection, help distribute traffic loads to adjacent girders, and help the superstructure to resist lateral loads such as wind. In curved or skewed superstructures, cross-frame loads create lateral bending moments in the girders that are resisted primarily by the girder flanges. In typical curved-girder analysis, the total tip stress in the girder flanges is the sum of the stresses caused by the vertical bending moments and the lateral bending moments.

However, there can be a third source of stress in the girder flanges that is not captured by standard computerized analysis. Unless they are specifically detailed to avoid this condition, most curved steel girders are not vertical (plumb) in their final condition. Curved girders that are fabricated to be erected plumb in the no-load or steel dead load only condition will tend to rotate into a position in which their webs are not plumb in the final condition. If the webs are very close to plumb (which is the case for many girders with large radii or shorter spans or both), this will have little effect on the girder stresses. But as the magnitude of the web out-of-plumbness increases, the lateral component of the vertical bending moment can become a significant component of the girder stresses, possibly increasing the flange tip stress by 5% or more. This lateral component of the vertical bending moment is a secondary effect caused by the true deflected shape of the superstructure and is not captured by typical curved-girder analysis methods. A separate method for quantifying the lateral component of the vertical bending moment is needed. This is often overlooked by designers, but it is beginning to be recognized by some industry agencies. The AASHTO–National Steel Bridge Alliance steel bridge collaboration document, *Guidelines for Design for Constructability* (3), directly addresses the additional stresses caused by out-of-plumb girders and directs the designer to evaluate them.

Consideration of Out-of-Plumb Effects

Given the girder depth and the vertical deflections of the five girders, the global rotations of the superstructure were calculated at tenth points along each span. These rotations were used to calculate lateral bending factors, which quantified the lateral components of the positive vertical bending moments. In effect, the spreadsheet resolves the plumb vertical bending moment into a lateral component and a component aligned with the out-of-plumb web. See Table 3 for an example spreadsheet and Figure 3 for a sketch explaining the calculations. Only the steel dead load and noncomposite deck conditions were included in the calculations, since the additional flange stresses induced on the composite section are minimal.

The products of the lateral bending factors and the vertical bending moments from BSDI-3D were the additional lateral bending moments caused by the out-of-plumb nature of the girder webs. A second spreadsheet incorporated these additional lateral moments into the girder stress checks and calculated new performance ratios for critical locations along each girder. As seen in Table 4, additional flange tip stresses as high as 1.61 ksi (11.10 MPa) were seen, which caused a 4.6% increase in the girder stresses calculated by BSDI-3D. An optimized girder design will not typically have sufficient reserve capacity to accommodate a stress increase of this magnitude.

The proposed approach for incorporating the additional lateral flange moments caused by the out-of-plumb webs ensures that the girder stress checks include all relevant stresses. The spreadsheet method will increase the final girder lateral flange bending moments, reducing the factor of safety at each checked location along the girders. A designer who uses a program like BSDI-3D to perform girder stress checks must

TABLE 3 Calculation of Lateral Bending Factors

Location Along Span	Steel Dead Load Only: Vertical Deflection (in.) G1	G5	Difference in Vertical Deflections (in.)	Lateral Bending Factor
Span 1				
0.0	0.00	0.00	0.00	0.00000
0.1	0.02	−0.27	0.29	0.00057
0.2	0.10	−0.46	0.56	0.00109
0.3	0.26	−0.55	0.81	0.00158
0.4	0.47	−0.52	0.99	0.00193
0.5	0.71	−0.38	1.09	0.00213
0.6	0.91	−0.18	1.09	0.00213
0.7	0.96	0.02	0.94	0.00184
0.8	0.84	0.15	0.69	0.00135
0.9	0.51	0.15	0.36	0.00070
1.0	0.00	0.00	0.00	0.00000
Span 2				
0.0	0.00	0.00	0.00	0.00000
0.1	−1.03	−0.49	−0.54	0.00105
0.2	−2.29	−1.19	−1.10	0.00215
0.3	−3.52	−1.95	−1.57	0.00307
0.4	−4.40	−2.54	−1.86	0.00363
0.5	−4.87	−2.89	−1.98	0.00387
0.6	−4.85	−2.93	−1.92	0.00375
0.7	−4.30	−2.62	−1.68	0.00328
0.8	−3.26	−2.00	−1.26	0.00246
0.9	−1.80	−1.11	−0.69	0.00135
1.0	0.00	0.00	0.00	0.00000

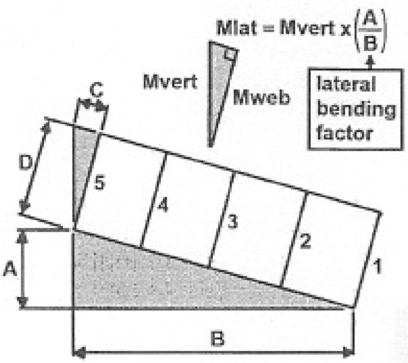

FIGURE 3 Derivation of lateral bending factors.

remember to allow some additional conservatism in the BSDI-3D girder design.

PENNSYLVANIA STATE UNIVERSITY MONITORING PLAN

Research is being conducted at the Pennsylvania State University into the behavior of curved-girder bridges during construction. It is the goal of the research to provide insight into the actual behavior of these structures during construction to make it possible to develop more accurate and effective construction procedures in the future. This will be accomplished through instrumentation and monitoring of Bridge 207 during all phases of construction, including girder shipment, superstructure erection, and concrete deck placement. The work focuses on axial stresses in specific cross-frames, warping stresses of the girder flanges, and global deformations and rotations of the steel superstructure.

To achieve the objectives of the research, the following tasks were completed:

- A preliminary analysis of the structure was performed with finite element models.
- Instrument locations were selected, and vibrating wire instruments were installed at critical locations on the girders and cross-frames.
- The structure was closely monitored during all construction activities, and a detailed construction time line was documented. A time line was necessary to fully understand the condition of the structure and loading scenarios present at any given point in time.
- A photogrammetry study of the bridge was conducted to monitor changes in the geometry of the structure through construction activities.
- Grillage models were developed considering actual geometry, erection sequence, and deck placement procedures for comparison with field data.

Constructibility Problems

Most problems with the construction of curved-girder bridges occur during superstructure erection (*4*). Girder and cross-frame fit-up problems occur for several reasons. Frequently, proper consideration is not given to the girder deflections and rotations at each stage of erection. Current AASHTO guide specifications now require consideration of the effects of curvature during fabrication, shipment, erection, and deck placement but provide little guidance for the designer or contractor to do so (*5*). As a result, temporary supports and lateral bracing necessary for the construction of bridges curved in plan are often inadequate. This leads to girder and cross-frame fit-up problems and difficulties achieving desired final geometry. Numerous studies have indicated that three-dimensional finite element models can accurately predict the behavior of these structures, yet grillage analogies are often used by contractors to develop erection sequences because they are simpler to construct and require significantly less computing power. The ability of these models to predict the effect of curvature on stresses and deformations during construction is addressed in this research.

A second problem common with these structures is inconsistent detailing of cross-frames (*6*). The web of a curved girder cannot be plumb in the no-load, dead load, and final conditions because of rotation of the cross-section during construction. When the bridge designer fails to recognize this crucial fact, inconsistent detailing can occur. For example, if a bridge is designed for the girder webs to be plumb under the self-weight of the superstructure, the cross-frames cannot be detailed for plumb webs in the no-load condition. These errors can lead to fit-up problems and geometric discrepancies.

Preliminary Analysis

To assist with instrument placement, three-dimensional finite element models were generated by using SAP2000, a commercially available structural analysis software package. The models were used to determine the magnitude and locations of maximum positive and negative

TABLE 4 Additional Lateral Bending Stresses Caused By Out-of-Plumbness

		Top Flange		Bottom Flange	
	Location Along Span	Maximum Additional Lateral Bending Stress	Percentage Increase in Lateral Bending Stress	Maximum Additional Lateral Bending Stress	Percentage Increase in Lateral Bending Stress
Girder G5, Span 1	0.4L	0.58 ksi (4.00 MPa)	1.9	0.47 ksi (3.24 MPa)	1.0
Girder G5, Span 2	0.4L	1.61 ksi (11.10 MPa)	4.6	0.87 ksi (6.00 MPa)	1.9
Girder G1, Span 1	0.7L	0.88 ksi (6.07 MPa)	1.8	0.28 ksi (1.93 MPa)	0.8
Girder G1, Span 2	0.4L	1.51 ksi (10.41 MPa)	3.3	0.81 ksi (5.58 MPa)	1.6

NOTE: Additional stresses in Table 4 are listed for steel dead load plus noncomposite deck condition.

vertical bending in both spans. Changes in concrete deck properties caused by staged deck placement were considered in the model by modifying the material properties of the concrete. For wet concrete during deck placement, material properties for the deck were given the unit weight of concrete but negligible stiffness to prevent composite behavior. Values for the modulus of elasticity were estimated and changed as applicable for different stages of deck placement. Deck placement was assumed to be accomplished in three stages (see Figure 1 for span locations), which included the positive moment region in Span 1, the positive moment region in Span 2, and the negative moment region.

The model was used to obtain deflections and flange stresses caused by the self-weight of the steel superstructure and the combined weight of the steel superstructure and concrete deck along the length of all five girders. Results were used to determine optimal locations for vibrating wire strain gauges and tiltmeters.

Instrumentation Plan

A total of 65 vibrating wire strain gauges and four vibrating wire tiltmeters were used to monitor the study bridge during construction. The vibrating wire instruments were placed at anticipated points of maximum positive bending in Span 1 and Span 2, the point of maximum negative bending at the pier, and Abutment 2. More vibrating wire instruments were placed on the exterior girders and cross-frames since these are the critical elements within a curved-girder system.

Normal strains were measured at the anticipated points of maximum positive and negative bending. Geokon Model VK-4100 vibrating wire strain gauges, shown in Figure 4, were used for this purpose. The instruments have a gauge length of 2 in. (51 mm), a range of 2,500 $\mu\epsilon$, and a sensitivity of 0.5 to 1.0 $\mu\epsilon$. The gauges were welded to the top and bottom flanges of the girders, 1 in. (25 mm) from the flange tips. To measure warping stresses, gauges were needed at both flange tips.

Rotation instruments were used to measure longitudinal rotations at Abutment 2 and radial rotation of the entire superstructure near the point of anticipated maximum positive bending in Span 2. Geokon Model 6350 vibrating wire tiltmeters were used. These instruments have a range of $\pm 10°$ and a resolution of 10 arc seconds. The tiltmeters were affixed to the girders with a bracket bolted to the web near the noncomposite neutral axis.

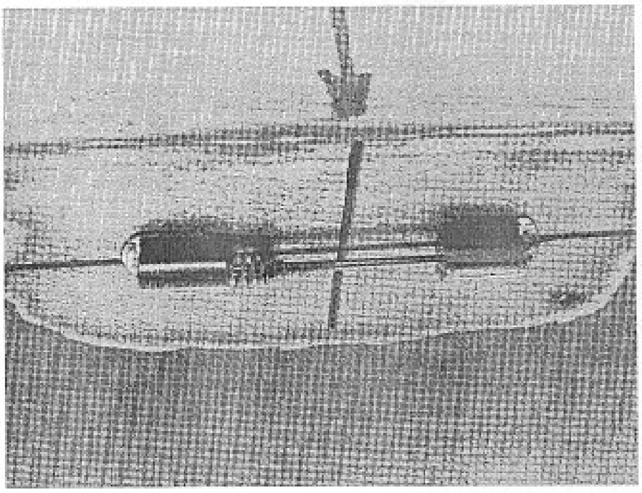

FIGURE 4 Geokon Model VK-4100 vibrating wire strain gauges.

Installation and Monitoring Plan

Vibrating wire instruments were installed on the girders at both the fabrication yard and the project site. Data were acquired manually in the fabrication yard and during girder erection by using a portable vibrating wire readout box. On completion of girder erection, a 64-channel data logger and control module was installed that acquired data continuously throughout remaining construction activities. These instruments were hardwired on installation of the metal decking but before placement of any temporary forms for deck placement.

The girders were blocked to their vertical curvature and camber in the fabrication yard, and strain gauges in the positive moment region of Span 2 were installed. Benchmark readings were taken with the portable readout box. These instruments were monitored before and after girder shipment and during girder erection. These gauges were the primary instruments used for the superstructure erection study. The data from these instruments during girder erection will be used to report the effect curvature has on girder warping stresses during construction and assess the ability of a grillage model to predict these effects.

Remaining top flange strain gauges were installed while the girders were blocked to curvature and camber at the project site. Since the bottom flange was not accessible at this time, bottom flange strain gauges in Span 1 and at Pier 1 were installed once the girders were in place. Because of time restraints, tiltmeters and cross-frame strain gauges also were not installed until completion of girder erection. These instruments were primarily used for the deck placement study along with previously installed gauges.

The structure was closely monitored during erection of the superstructure. Readings were taken by using the portable readout box at the end of each working day, and a detailed time line of the actual erection procedure was recorded. This was completed to evaluate the effect of curvature on the partially erected superstructure as components were added to the structural system. The contractor used a single girder erection procedure, working from the interior girder, G5, to the exterior girder, G1. The shorter span, Span 1, was erected in its entirety before Span 2 was constructed. The contractor used several temporary shoring towers, temporary tie-downs, and lateral wind bracing during girder erection and deck placement to stabilize the system.

Another set of benchmark readings was taken with the portable readout box on completion of girder erection and placement of deck pans to distinguish strains caused by the erection sequence from strains caused by deck placement procedures. At this time the data-logger was installed. Readings were taken at half-hour intervals through placement of deck steel and forms and 5-min intervals during placement of the concrete deck to filter out the effects of vibrations from construction equipment on the gauge readings. Again, a detailed time line of the actual deck placement procedure was recorded. The deck was placed in three stages, as anticipated. The positive moment region in the shorter span was placed first, followed by the positive moment region in the longer span 5 days after the previous pour. The negative moment region was the last section to be placed to minimize cracking of the deck caused by its self-weight. This occurred 3 days after the second pour. Each pour took approximately 6 h, and bulkheads were oriented radially near the anticipated dead load inflection points.

Photogrammetry Study

Greenman-Pederson, Inc., conducted photogrammetry scans by using Cyrax Laser Technology. Scans were completed at three critical stages during construction of the structure, including completion of girder erection, placement of deck steel, and placement of the concrete deck. Each scan produced a three-dimensional grid of the entire structure and will be used to determine translations and rotations along the entire length of all five girders and the global twisting effect of the unbalanced spans. It will be possible to measure out-of-plumbness of the girder webs by using these data. These data will also be used to determine if the as-built geometry of the bridge relates well with the design geometry. All this will be compared with grillage analogies of the structure to determine if the models can accurately predict deflections of Bridge 207 during construction.

PRELIMINARY MONITORING RESULTS

Data from the curved-girder study are still being collected and assessed. The vibrating wire strain gauges and tiltmeters worked well for monitoring the curved-girder structure over an extended period, and data acquisition and reduction were relatively simple. Future field studies should focus on monitoring the girders along an unbraced length rather than at critical locations only to evaluate the effect of curvature and construction on the distribution of flange warping stresses as well as magnitudes at critical locations.

Preliminary comparisons of grillage models and field data have been conducted for flange vertical bending stresses and vertical deflections during construction, but flange warping and lateral bending stresses have not been evaluated. It was determined that curvature had a measurable effect on vertical bending of the girders, and the grillage model in SAP2000 predicted vertical bending stresses throughout girder erection and deck placement reasonably well. Vertical deflection comparisons also showed reasonable agreement; however, grillage model predictions were consistently nonconservative when compared to deflections measured in the field for all five girders. Figures 5 through 8 show comparisons of field data and the grillage model predictions.

CONCLUSIONS

Curved steel plate girder bridges are complicated three-dimensional systems. The girder curvature causes deflections and internal forces that sometimes differ from what would be seen in a tangent structure of similar size. An asymmetrical curved span arrangement can exhibit even more complex behavior. An understanding of the rolling behavior of curved girders is crucial for engineers who analyze and design these structures. The additional lateral flange bending moments gen-

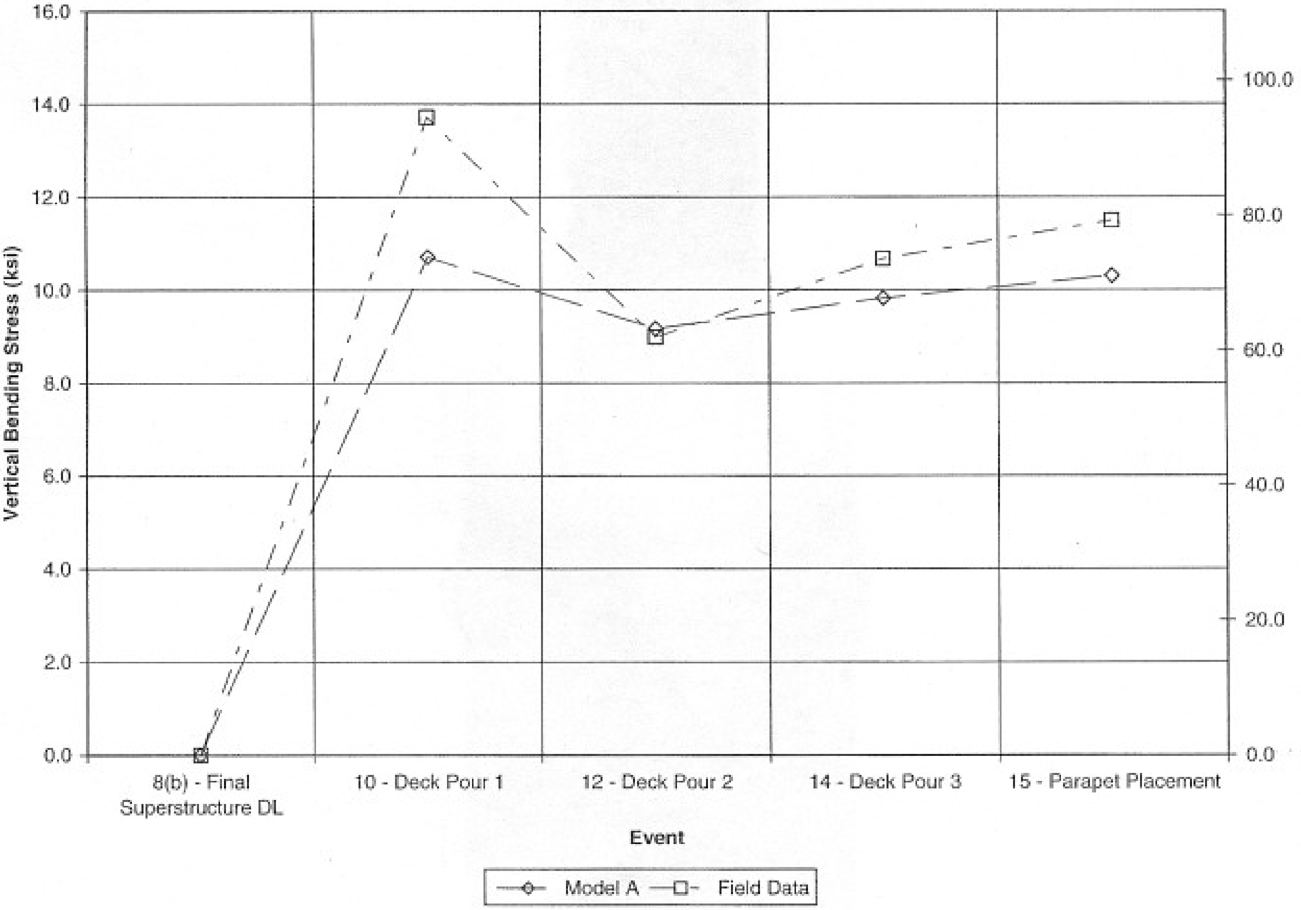

FIGURE 5 Girder G1 bottom-flange vertical bending stresses during deck placement.

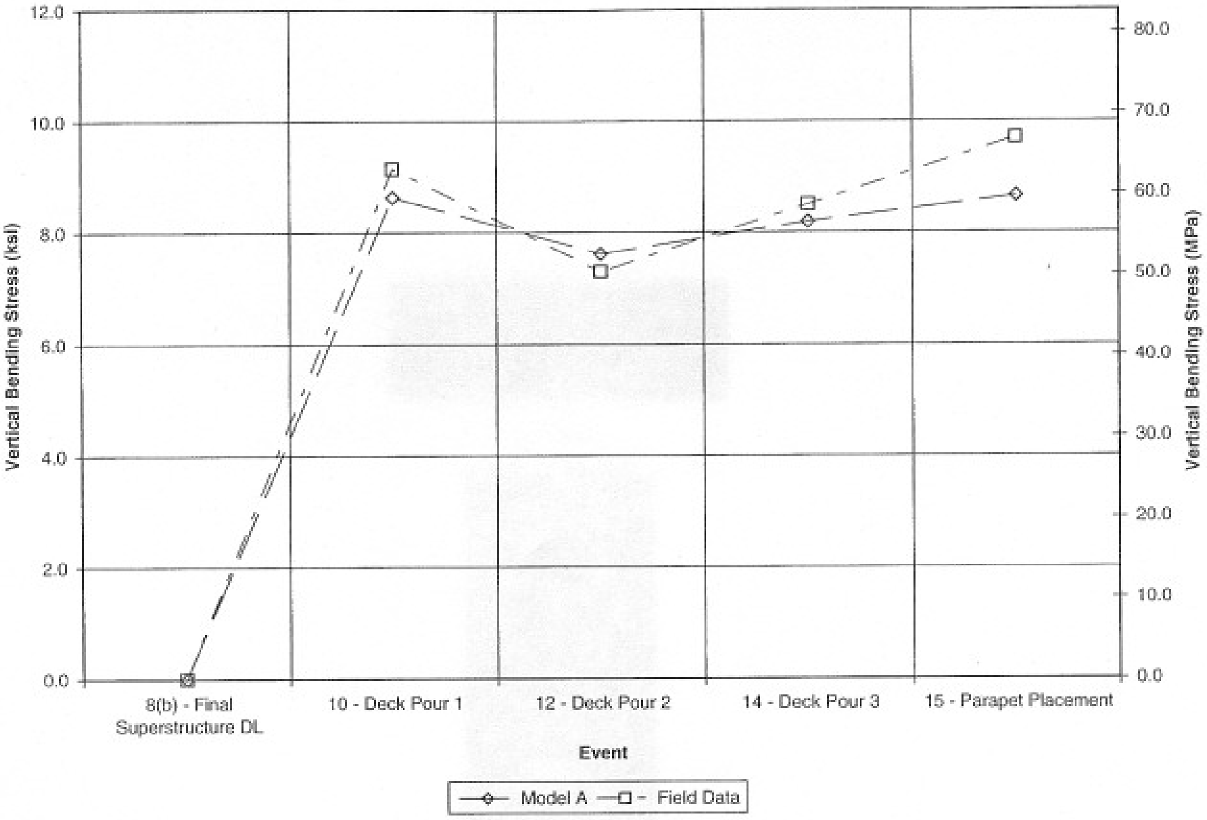

FIGURE 6 Girder G5 bottom-flange vertical bending stresses during deck placement.

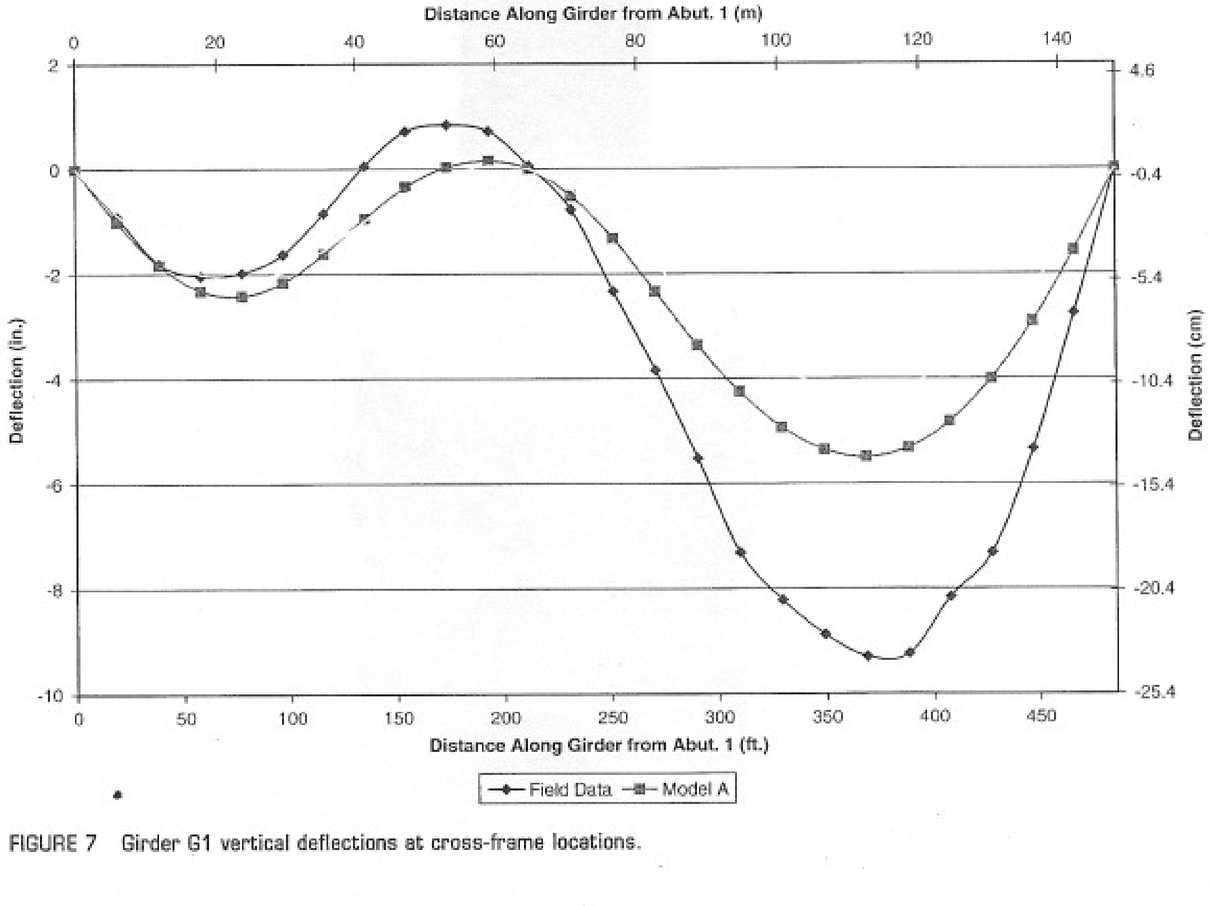

FIGURE 7 Girder G1 vertical deflections at cross-frame locations.

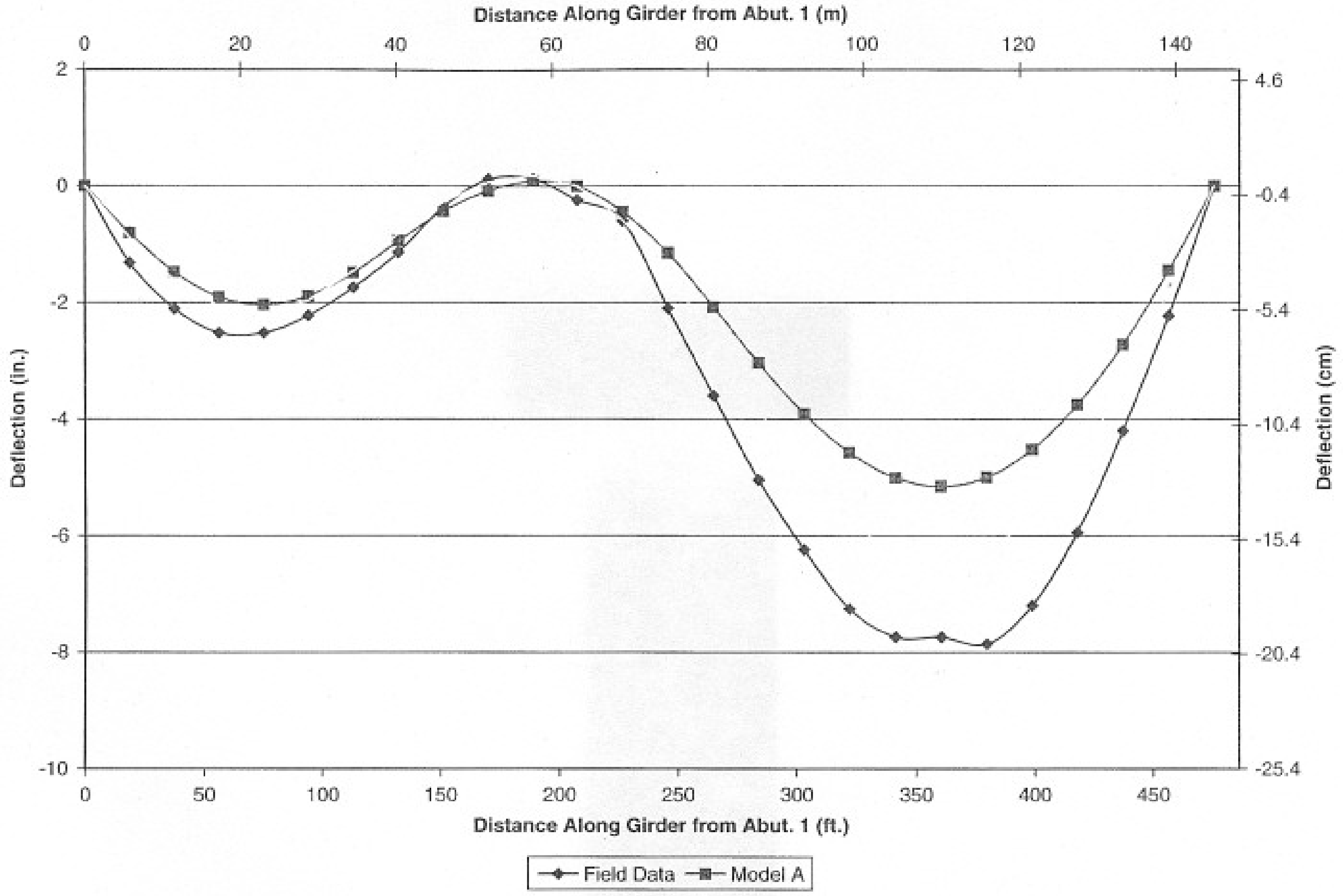

FIGURE 8 Girder G5 vertical deflections at cross-frame locations.

erated by the out-of-plumb nature of curved girders should be considered by the designer. Field data that are being collected on Bridge 207 in Centre County, Pennsylvania, will be used to describe the complex internal stresses in these structures at all stages of erection. This research will also be used to evaluate the accuracy of curved-girder grillage models and three-dimensional finite element models.

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